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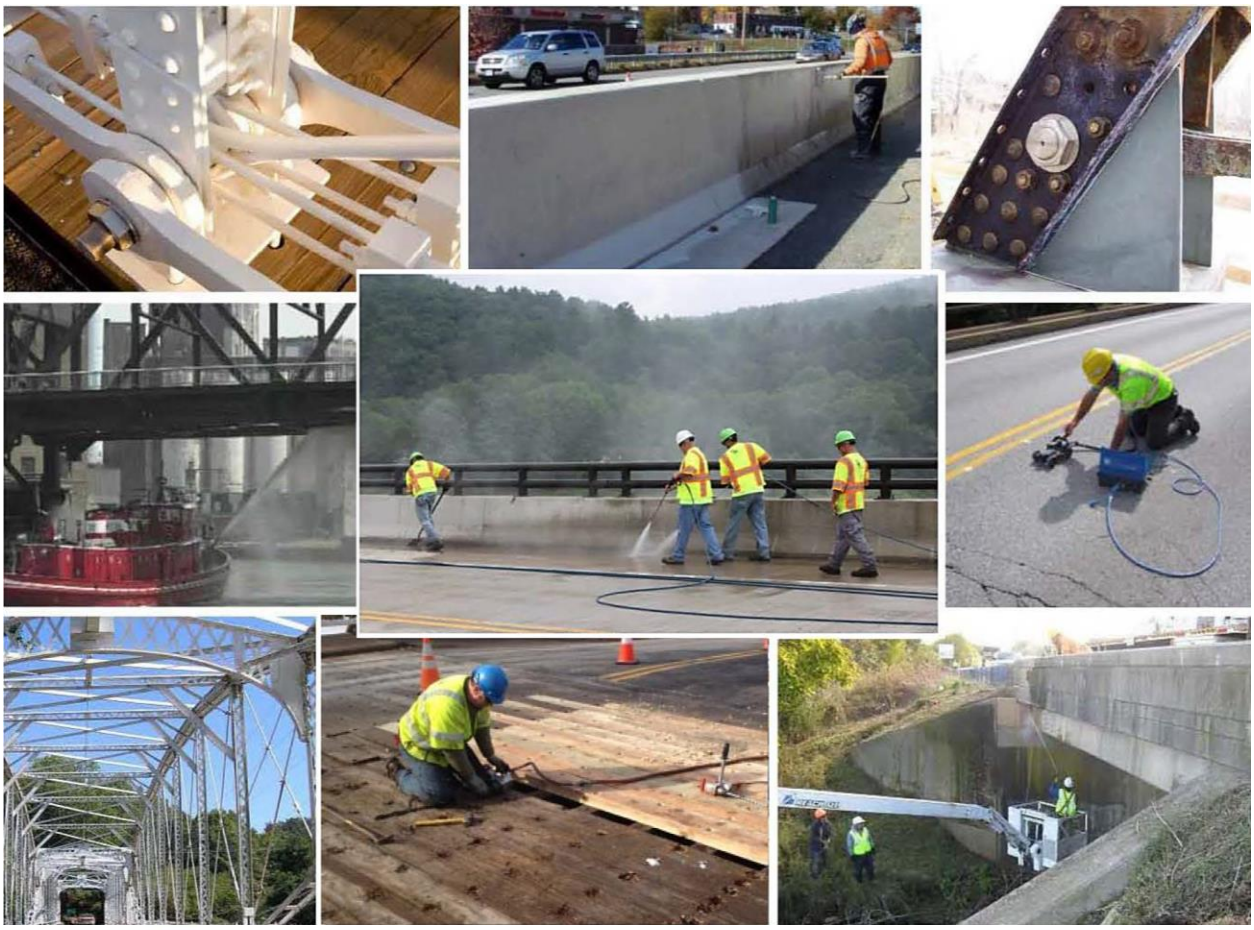
Bridge Maintenance Reference Manual



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Chapter 1 - Introduction to Bridge Maintenance

1.1 Fundamentals of Bridge Maintenance

1.1.1 Target Audience

This manual provides fundamental information pertaining to bridge maintenance for those new to and for those advancing in their roles and responsibility within bridge maintenance. The manual serves as a refresher and/or a reference for those currently working in bridge maintenance. Program managers will find this manual useful in a review of an existing bridge maintenance program or when implementing new activities in existing programs.

The manual is designed to assist bridge maintenance workers and managers in addressing most common types of bridge distress. Practical procedures for preventive maintenance on highway bridges and culverts are outlined. This manual is not meant to be all-inclusive and does not rule out other maintenance procedures. It is a source of information for those responsible for maintenance and preservation of highway bridges and provides an understanding of strategies and methods of bridge maintenance programs. The manual is not intended to supplant existing agency guidelines or policies, but to be used as a source of information when the need arises. The manual is not a substitute for consulting an engineer with the appropriate qualifications to determine proper repair methods.

This manual is of value to engineers, program managers, supervisors, foremen, and field crews. Engineering consultants, academics, and industry representatives will also find this manual to be a valuable resource for maintenance techniques, materials and methods, concepts, strategies, and guidance.

1.1.2 Other Resources

The Federal Highway Administration (FHWA) and its training division, the National Highway Institute (NHI), offer many resources that are beneficial to those involved in bridge maintenance. These include:

- NHI Course 130088 Bridge Construction Inspection
- NHI Course 130054 Engineering Concepts for Bridge Inspectors
- NHI Course 130055 Safety Inspection of In-Service Bridges
- NHI Course 134029 Bridge Maintenance Training
- NHI Course 134108G TCCC Plan Reading: Bridge Plans (Web Based)

These and many other related courses can be found at

https://www.nhi.fhwa.dot.gov/default.aspx.

1.1.3 History of Bridge Maintenance

In order to meet the city of Rome's water needs, a system of 11 aqueducts was constructed between 312 B.C. and 226 A.D. Portions of this system, like the section of the Aqua Claudia pictured in Figure 1.1, remain standing to this day as a result of proper maintenance. Historic records indicate crews flushed the pipes of sediment, repaired breaches, and removed calcium carbonate build-up on a periodic basis. Aqueduct designers included access points at regular

intervals for inspection and maintenance activities. Throughout the evolution of bridge building, maintenance has played a role in keeping highway structures in service. As designers continue to incorporate modern codes and materials into bridge design, bridge maintenance continues to develop and implement new materials, methods, and strategies to keep existing bridges in service.

Figure 1.1 Section of the Aqua Claudia, South of Rome

1.2 The Bridge Maintenance Unit

1.2.1 Organizational Structure

Most transportation agencies have a Central Office that performs administrative functions in support of the District (or Regional) offices. The people that repair and maintain the highway system work in the Districts.

Many Departments of Transportation (DOT) are organized by functional area. Units involved with the design of construction projects, developing standard specifications, and updating the Qualified Products List, for example, are commonly housed in a division separate from units responsible for snow removal, highway maintenance, and fleet management. District operations are overseen by administrative counterparts in the Central Office. Figure 1.2 and Figure 1.3 illustrate two different ways in which a DOT might incorporate the bridge maintenance function into their organization.

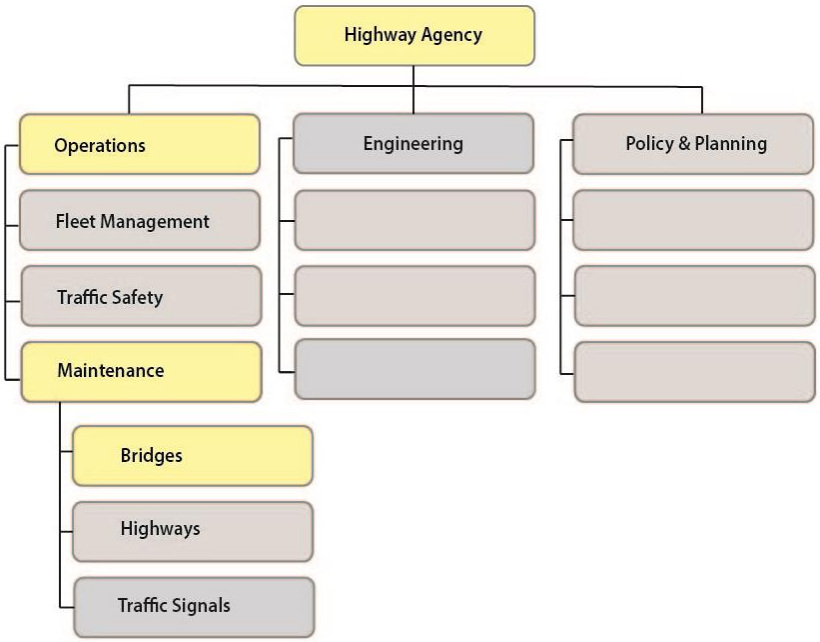

Figure 1.2 Organization Chart with Bridge Maintenance as Part of Operations

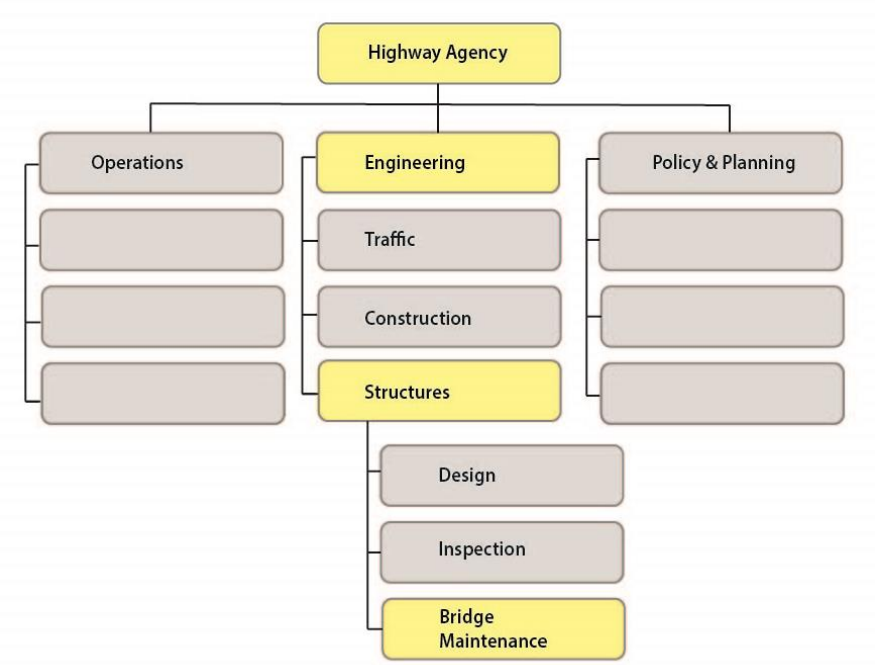

Figure 1.3 Generic Organization Chart with Bridge Maintenance as Part of Engineering

Organization may be less formal in smaller agencies. A single engineer may serve as bridge designer and bridge maintenance engineer. Some agencies are organized without specific job titles. In this instance, bridge maintenance is performed by the same crews that maintain pavement and other roadside assets such as drainage, guardrail, signs, lighting, etc.

Some examples of ways in which transportation agencies have integrated the bridge maintenance function within their organization are provided below:

- A unit in the Central Office controls all the bridge maintenance work for the entire agency, and all the work is contracted out.
- A unit within the Central Office provides technical advice or guidance. The District performs maintenance work.
- A unit in Central Office is responsible for major repairs and rehabilitation and the Districts are responsible for minor repairs, preventive and other ordinary maintenance activities.
- The Maintenance group in the Central Office works with the maintenance units in the District offices. The Central Office Structures group provides technical support to the maintenance units.
- The Structures group in the Central Office determines the bridge maintenance program and designs the repairs. Districts conduct the contract administration including the inspections and final acceptance of the work.
- One unit in the Central Office is responsible for the design and maintenance of highway structures. The agency does not have District offices.

There isn't a “best” place for bridge maintenance within an organization. Whether by function, personnel, or asset, each unit and the people working in it become effective through strong communication and support within and across organizational lines.

1.2.2 Developing the Work Plan

Just like there are many ways for a DOT or other transportation agency to integrate the bridge maintenance function within their organizations, there are also many ways in which these same organizations develop their bridge maintenance work plans. Figure 1.4 illustrates how one state’s DOT handles the decision-making process in the development of its work plan. In this example, the Regional Bridge Maintenance Engineer (RBME) determines if repairs are necessary, and if the repairs can be performed by state forces. It also shows how the Maintenance Management System (MMS) and Bridge Management System (BMS) are incorporated into the process.

Most bridge repair needs are identified through a periodic bridge inspection program. Bridge inspectors provide detailed information and photographs of existing bridge conditions. The inspection report and bridge inventory data are the primary sources of information for an initial determination of the various maintenance and preservation needs of a bridge. When a bridge has been identified as needing work, more detailed information is gathered during a field visit by maintenance personnel.

In a few agencies, inspectors are required to provide a list of maintenance actions and repair quantities to the Bridge Maintenance Unit. In addition to the inspection process, some agencies include periodic review of the bridge by maintenance personnel.

Agencies use Bridge Management Systems (BMS) to identify and prioritize bridge needs. This is typically a Central Office function. Districts compare BMS lists of maintenance needs to Districts’ internal lists of needs. The Districts are usually given the authority to modify the initial BMS list and the scope of work whether generated by the Central Office or District/Local office is typically adjusted by the staff responsible for the bridge maintenance program.

Some maintenance actions are done on a fixed cycle. Candidates for cyclical maintenance are determined through simple queries of the bridge management system. For instance, all bridges with concrete decks that have not been sealed in the last 10 years might be candidates for deck sealing. Deciding if it is worthwhile to seal a deck that is scheduled for an overlay or replacement filters the list further.

Bridge needs can arise suddenly. Impacts to the superstructure by over-height vehicles, severe weather events, and punch-throughs of the deck are not uncommon and add to the work load of the Bridge Maintenance Unit.

Agencies have various systems to identify and relay urgent needs to the Bridge Maintenance Unit. Some agencies have instituted a flagging system in the bridge inspection process to alert bridge maintenance of an urgent repair need.

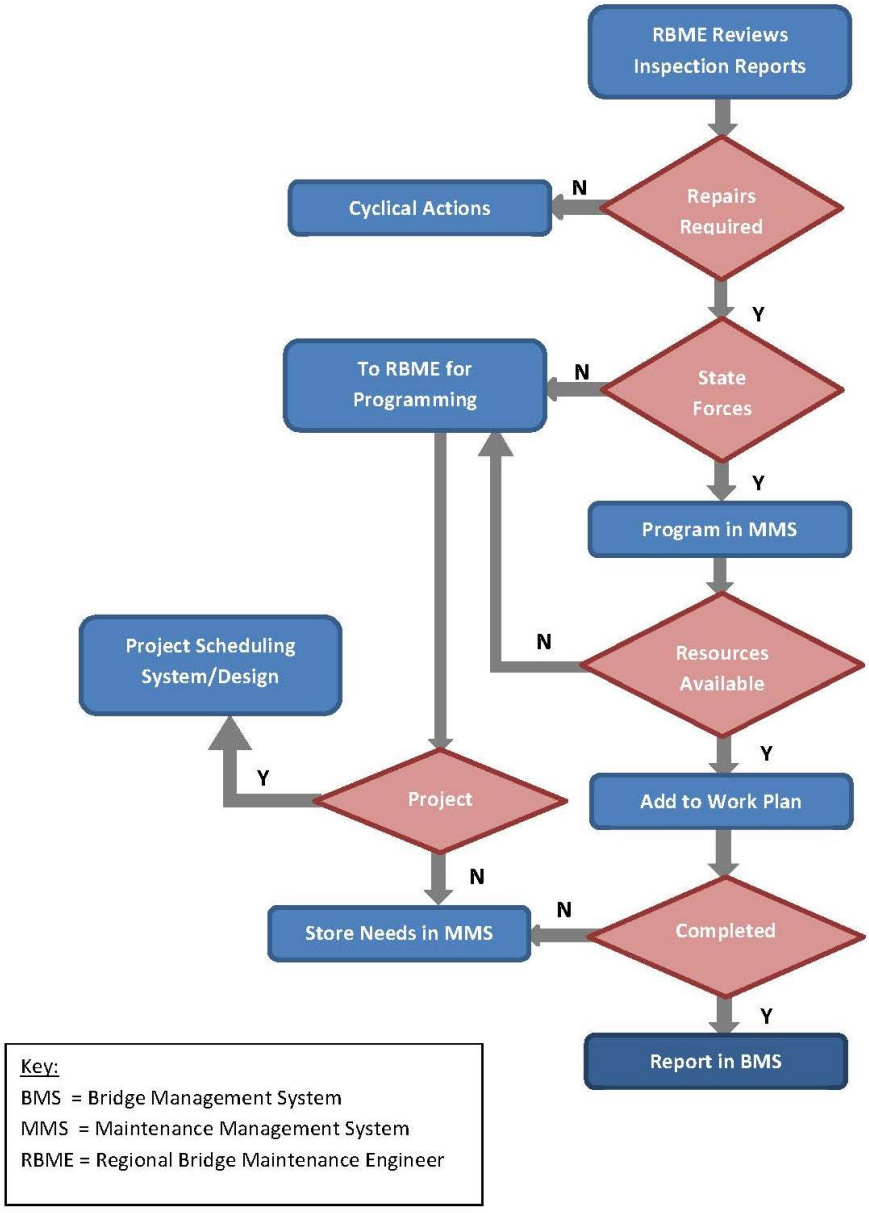

Figure 1.4 Example of Work Plan Decision-Making Process (Courtesy of New York State DOT)

1.3 When to Call the Engineer

Bridge maintenance staff may encounter conditions on bridges that require the attention of an engineer. Bridge inspectors inspect a bridge once every 24 months, but bridge conditions can change rapidly due to storms, floods, traffic impacts or overload, earthquakes, utility failures, and embankment erosion. It is possible that a bridge maintenance worker may have better access to view areas or conditions that were not visible to the bridge inspector during their routine inspections. In general, bridge maintenance staff should contact the engineer whenever any portion of the primary load path of the bridge is compromised. The Primary load path is the direction in which each consecutive load will pass through connected members. The sequence commences at the highest point of the structure working all the way down to the footing system, ultimately transferring the total load of the structure to the foundation.

Ultimately, the lowest structural member must be strong enough to support all members above it.

For most bridges the primary load path components are deck and slab elements, primary superstructure elements, and primary substructure elements. These components are discussed in the following sections.

1.3.1 Deck and Slab Elements

Decks and slabs transfer tire loads to the superstructure of the bridge. Any deck or slab conditions that could cause a full depth hole should be reported to the engineer. This may be heavily cracked concrete with intersecting cracks on top of the deck and underside that occurs within any 12 inch by 12 inch area or larger. These “deck punch-throughs” are also possible in steel grid decks that have cracked or missing grid pieces. In timber decks, failure to transfer tires loads most often comes in the form of cracking and rot.

1.3.2 Primary Superstructure Elements

The role of the superstructure elements of a bridge is to carry the loads transferred from the deck to the substructure. Superstructure elements are typically the elements of the bridge that span over obstructions or roadways. The following is an example listing of superstructure field conditions that warrant a call to the Engineer:

<u>Steel girders/ stringers/ floor beams/ truss members/ arched or cabled bridges</u>

- Any cracks found in primary members or welds to primary members such as girders, floor beams, pin and hangers, truss connections etc.
- Any section loss that exceeds 10 percent of the member thickness
- Bent, bowed, or distorted steel elements
- Broken cable elements or cable anchorages
- Missing or loose bolts, rivets, or broken welds
- Loss of bearing

<u>Concrete girders/ stringers and floor beams/ arches</u>

- Impact damage that bends or breaks reinforcing steel or prestressing strands
- Any material deterioration evidenced by dense crazed cracking
- Any loss of bearing

- Any fire damage that turns the concrete pinkish or produces spalling
- Cracking greater than 1/16 inch wide for prestressed and 1/8 inch wide for reinforced concrete particularly near midspan, or near supports
- Unsound concrete at or behind reinforcing steel

Timber girders/ stringers/ floor beams

- Any broken or split members
- Any rotting, crushing, or decay found near the ends of the members
- Any decay or fire damage that results in a 10 percent loss of the members section
- Loss of bearing
- Any broken or missing bolts or screws

1.3.3 Primary Substructure Elements

The substructure elements of the bridge take the loads from the superstructure and transfer the loads to the ground. Substructure elements include bent caps, columns, pier walls, abutments, footings, piles, trestles and towers. The following is an example listing of substructure field conditions that warrant a call to the Engineer:

Steel Substructure Elements

- Any broken or cracked members
- Any section loss that exceeds 10 percent of the member thickness
- Bent, bowed or distorted steel elements
- Missing or loose bolts, rivets or broken welds
- Erosion holes under or around bridge supports
- Buildup of soil in channels that restricts 20 percent or greater of the normal opening

Concrete Substructure Elements

- Impact damage that bends or breaks reinforcing steel or prestressing strands
- Any material deterioration evidenced by dense crazed cracking
- Any loss of bearing
- Cracking greater than 1/8 inch wide particularly in bent caps
- Erosion holes under or around bridge supports
- Any fire damage that turns the concrete pinkish or produces spalling
- Buildup of material in channels that restricts 20 percent or greater of the normal opening
- Cracking greater than 1/16 inch wide for prestressed and 1/8 inch wide for reinforced concrete particularly near midspan, over or near supports
- Unsound concrete at or behind reinforcing steel

Timber Substructure Elements

- Any broken or split members
- Any rotting, crushing or decay found near the ends of the members
- Any decay or fire damage that results in a 10 percent loss of the members section
- Any members that are not fully bearing

- Any broken or missing bolts or screws
- Erosion holes under or around bridge supports
- Buildup of material in channels that restricts 20 percent or greater of the normal opening

Culverts

- Distortion along the length of the culvert in excess of 15 percent difference from the design dimensions, or if the distortion has changed since the previous inspection.
- Buildup of material in culvert that restricts 20 percent or greater of the normal opening
- Erosion of the soil under the culvert or around the headwall and wingwalls
- A dense network of random crack indicating material deterioration
- Any fire damage that turns the concrete pinkish or produces spalling
- Any material deterioration evidenced by loss of section in the flow line

Other situations that warrant a call to an engineer

- Cracked steel or aluminum joint assemblies
- Buckled approach slabs
- Signs of approach slab settlement that exceeds 2 inches
- Signs of any bridge substructure settlement not documented in the bridge inspection report
- Exposure or near exposure of the bottom of spread footings that are not founded on rock
- Noticeable changes in the location of channels under bridges
- Before the depth of the wearing surface is increased or the depth of integral concrete wearing surface is decreased due to maintenance operations
- Before removing full depth portions of decks or large areas of concrete decks below the top layer of reinforcing steel

The lists above provide common situations that would warrant a call to an engineer. These lists are examples and are not comprehensive lists of conditions that warrant engineering review. If the stability of a bridge is in question, an engineer should always be contacted. It is a good practice to send a photo of the situation in question.

In the following chapters of this document, a call out is provided to flag situations when the Engineer should be called. This call out will look like:

When to Call the Engineer

Specifics on when to call the Engineer will be provided in this space.

1.4 Terminology

The FHWA defines a bridge as "a structure, including supports, erected over a depression or an obstruction, such as water, a highway or a railway, and having a track or passageway for carrying traffic, or other moving loads and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes." The purpose of this definition is to establish criteria to facilitate bridge inventory, inspection, and reporting requirements as outlined in the national Bridge Inspection Standards.

Bridge maintenance plans employ several categories of actions (see Figure 1.5). These categories, as defined in the FHWA Bridge Preservation Manual, are described in the followed sections.

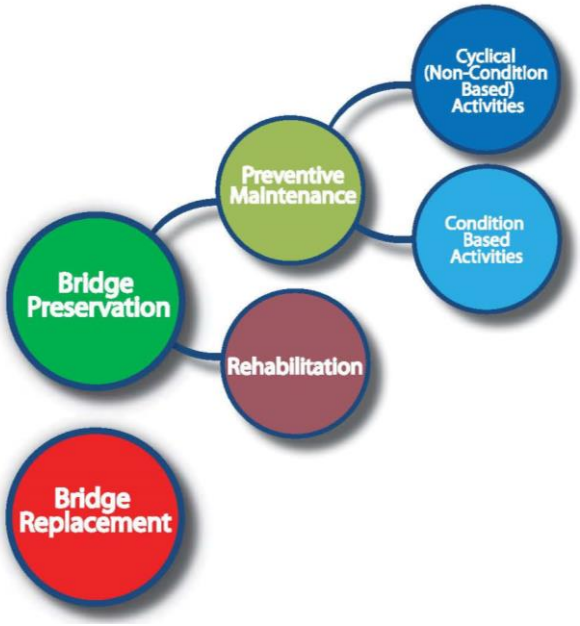

Figure 1.5 Bridge Action Categories as defined in FHWA Bridge Preservation Guide

1.4.1 Bridge Preservation

As defined by the FHWA Bridge Preservation Guide, bridge preservation includes "actions or strategies that prevent, delay or reduce deterioration of bridges or bridge elements, restore the function of existing bridges, keep bridges in good condition and extend their life". Preservation actions may be cyclical or condition-driven and may be preventive or restorative.

Bridge preservation actions can delay the need for costly replacement actions. Preservation actions are applied while the bridge is in good or fair condition, before the onset of serious deterioration. Bridge preservation encompasses preventive maintenance, repairs and rehabilitation activities. Functional improvement such as strengthening, widening, and raising vertical clearance are not considered preservation.

An effective bridge preservation program: 1) employs long-term strategies and practices to preserve the condition of bridges and to extend their service life, 2) has sustained and adequate resources and funding sources, and 3) has adequate tools and processes to ensure the appropriate treatments are applied at the appropriate time.

1.4.2 Preventive Maintenance

The American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Maintenance defines Preventive Maintenance as "a planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without substantially increasing structural capacity)."

Bridge owners apply preventive maintenance to elements or components of structures with significant remaining useful life. A part of bridge preservation, preventive maintenance is a strategy of extending useful life by applying cost-effective treatments to sound bridges (good or fair condition). The concept of preventive bridge maintenance suggests that a planned strategy of cost-effective treatment should be performed to keep bridges in good or fair condition, retard future deterioration, and avoid large expenses in bridge rehabilitation or replacements. This strategy is depicted in Figure 1.6, which shows conceptually how multiple preventive maintenance actions can be an alternative to multiple, costly rehabilitation.

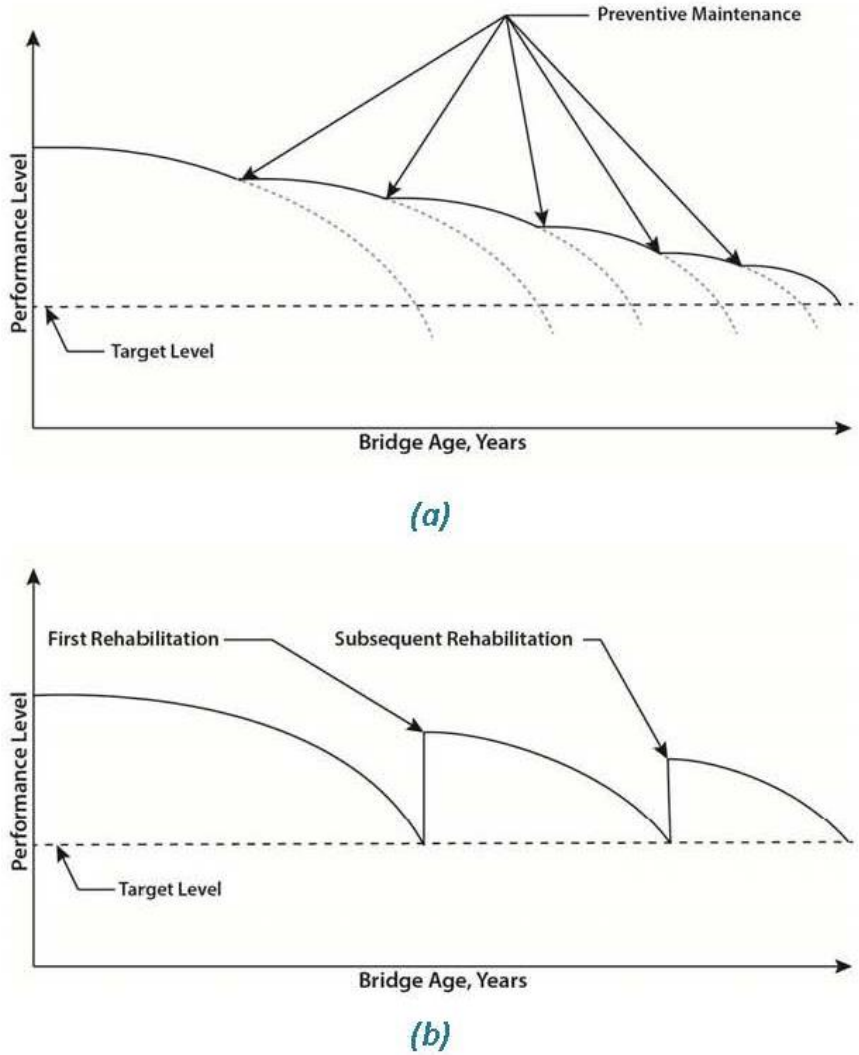

Figure 1.6 Bridge Life Extension by (a) Preventive Maintenance Only, and (b) Rehabilitation Actions with No Preventive Maintenance

Preventive maintenance includes cyclical actions and condition-based actions. Bridge rehabilitation is not considered a preventive maintenance action.

1.4.3 Cyclical Maintenance (Scheduled Maintenance)

Cyclical maintenance, also called scheduled maintenance, involves activities performed on a pre-determined interval or schedule. Cyclical maintenance prevents an element from becoming deficient, not to improve it. Like an oil change for a car, cyclical maintenance actions are included in the work plan because the element is due for maintenance.

Agencies have identified a number of activities to be performed on cycles. These activities are considered necessary, but do not alter the condition rating of the structure. Cyclical maintenance is considered to have a long term benefit to the bridge. The influence of cyclical maintenance on the service life of the element and the bridge, as a whole, is difficult to measure.

Examples of cyclical maintenance include bridge washing, bearing lubrication, concrete sealing, and slope clearing. Bridge washing, for instance, is done to remove de-icing chemicals from concrete elements and superstructure, flush road grit from expansion joints, and flush the drainage system. Some bridge owners wash bridges annually, other owners set a two year interval.

The cycle for sealing concrete is typically based on the service life projection of the concrete sealer specified by the agency. For example, some agencies seal their concrete bridge decks every three or five years. Sealing concrete bridge decks is usually more frequent than sealing substructures because the deck, as a riding surface, abrades at a quicker rate than the substructure elements.

1.4.4 Corrective Maintenance and Repairs

Corrective maintenance actions are repairs. Elements needing corrective maintenance are those that have failed or are anticipated to fail. Some repairs, such as expansion joint repair and deck patching, are minor repairs. Major repairs are larger in scope and require a substantial effort to restore deteriorated components to a safe and serviceable condition. This includes work such as cap beam/pier column repair, wing wall replacement, structural concrete repairs, and steel beam repairs. It is important to consult with an engineer for the analysis and design of any major repair.

Selection of the correct repair technique can depend on knowing the cause of a deficiency and not just symptoms. If the cause of a deficiency is understood, the correct repair method will be selected, and the repair should be successful. A general procedure to follow for designing and executing a repair involves the evaluation and determination of the cause of the deficiency and the methods and materials to be used in the execution of the repair.

1.4.5 Critical Findings

National Bridge Inventory (NBI) regulations require agencies to monitor and address critical inspection findings in a timely manner. Critical findings are urgent. Critical findings are different from other work requests from the bridge maintenance and inspection offices. Chapter 2 of this manual goes into detail on nationwide bridge inspection procedures.

1.4.6 State of Good Repair

A State of Good Repair refers to a condition in which the existing physical assets, both individually and as a system, are functioning as designed within their useful service life and are sustained through regular maintenance and replacement programs. For bridges, the state of good repair means that physical conditions of bridge elements, components, and entire networks of bridges allow bridges to function as designed and are sustained through regular maintenance, preservation, and replacement activities.

1.4.7 Service Life

The service life of an asset is the total period during which the asset remains in use. Maintenance can extend service life.

1.4.8 Design Life

The design life of a structure is the estimated time that the bridge is expected to remain in service. Design life differs from service life as design life is a theoretical number, whereas service life is an actual number. In some cases, the service life of a structure can exceed the design life. Without preventive or corrective maintenance, the design life of a highway bridge may not be reached.

1.4.9 Rehabilitation

Rehabilitation of a bridge involves a significant amount of work to restore structural integrity, to correct major safety concerns, and in some projects to increase capacity of a bridge. Rehabilitation work can be done on one or multiple elements. Bridge rehabilitation is considered to be a bridge preservation action by some agencies. However, as stated by FHWA, "functional improvements such as adding a travel lane or raising vertical underclearance, while often considered as rehabilitation are not considered preservation". (FHWA, *Bridge Preservation Guide*, 2011, pg. 9).

Examples of bridge rehabilitation include but are not limited to: deck replacement, superstructure replacement, and pier replacement. These projects typically require significant engineering resources, long completion schedules, and considerable funding.

1.4.10 Replacement

As defined by FHWA, replacement is the "total replacement of a structurally deficient or functionally obsolete bridge with a new facility constructed in the same general traffic corridor." Bridge replacement is not considered a maintenance activity. This definition was provided by FHWA for management and administration purposes of the former Highway Bridge Program Fund. This program was later consolidated with many other programs in MAP-21.

1.5 Benefits of Preventive Maintenance and Consequences of Deferred Maintenance

1.5.1 Balanced Approach

Studies (e.g., http://www.fhwa.dot.gov/Bridge/management/index.cfm) have shown that a bridge preservation strategy that employs the right treatments at the right times is the most cost-effective strategy for the management of highway bridges. It costs less to maintain bridges

in good condition than to maintain them in a deteriorated condition. Applying the wrong treatments or deferring maintenance altogether, leads to a major rehabilitation or possible complete replacement. Rehabilitation projects and replacement projects are costly.

Bridge repairs and rehabilitation improve the condition of the bridge. Each repair adds to the length of time the bridge will remain in service. As the structure ages further, the condition rating drops and soon another repair or rehabilitation becomes necessary. Eventually, the deterioration becomes extensive and the structure has to be replaced.

During the life of a bridge, some elements such as expansion joints and approach slabs are repaired several times, each time improving the element condition but not the overall bridge condition rating.

More extensive rehabilitation, such as deck or superstructure replacement have a greater effect on the bridge condition ratings, but are done less frequently. The deck may be replaced once during the life of the bridge. A second deck replacement project is not common as too many other elements of the bridge may also require replacement. At that point the service life of the bridge is reached. The bridge becomes a replacement candidate.

1.5.2 Funding Bridge Preservation

Bridge managers look to address the needs of all bridges in their inventory. They balance investments in new bridges, rehabilitation, and preventive maintenance activities. Just as using all available funds for bridge replacements would neglect the preservation of existing bridges, using all resources on preventive maintenance would ignore more significant needs, such as bridge replacements and rehabilitations.

Decisions on funding bridge preservation require clear objectives and measureable goals and are based on needs. An objective could be as simple as:

> Implement timely preservation treatments on structurally sound bridges, thereby extending their useful life.

This could be measured by determining a percentage of bridges in a state of good repair, based on condition ratings. Goals could be expressed as annual accomplishments in sealing decks or painting bridges on a set cycle. Needs would be based on an unbiased data collection process that can be imported into a management system to help prioritize. Such management systems are discussed in Chapter 2.

1.6 Chapter 1 Reference List

1. AASHTO. *AASHTO Maintenance Manual for Roadways and Bridges.* 4th Edition, Washington, D.C.: American Association for State Highway and Transportation Officials, 2007.
2. FHWA. *FHWA Bridge Preservation Guide*. Report Number FHWA-HIF-11042, Washington, D.C.: U.S. Department of Transportation, 2011.
3. FHWA. FHWA website showing overview of bridge management: http://www.fhwa.dot.gov/Bridge/management/index.cfm
4. NHI. National Highway Institute website: https://www.nhi.fhwa.dot.gov/default.aspx

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Chapter 2 - Bridge Maintenance Management

2.1 Bridge Inspection

2.1.1 Bridge Inspection Regulations

The National Bridge Inspection Program began more than 40 years ago following the collapse of the Silver Bridge over the Ohio River between West Virginia and Ohio. At that time, Congress directed the Secretary of Transportation (in cooperation with State Highway Officials) to establish:

1. National Bridge Inspection Standards (NBIS) to provide for the proper safety inspection of highway bridges, and
2. A program to train bridge inspectors to carry out the program.

As a result of this congressional direction:

1. The National Bridge Inspection Standards (NBIS) regulations were developed.
2. A bridge inspector's reference manual was prepared. (FHWA, *Bridge Inspector's Reference Manual (BIRM), 2012).*
3. A comprehensive training course, based on the reference manual, was developed.
4. The National Bridge Inventory (NBI) was established.

NBIS regulation, contained in the Code of Federal Regulations 23 CFR 650 Subpart C, sets the national standards for the safety inspection and evaluation of highway bridges. These are minimum standards and are often exceeded by more restrictive state or local standards. The NBIS apply to all highway bridges on public roads. The regulation specifies the following:

- Responsibilities of Bridge Inspection Organization
- Qualifications of personnel
- Inspection intervals
- Inspection procedures
- Bridge inventory

2.1.2 Bridge Inspection Reports and Ratings

2.1.2.1 Introduction to Bridge Inspection Reports

Bridge inspections are required on all bridges over 20 feet in total length on public roads. The Code of Federal Regulations defines the personnel qualifications, inspection types, frequencies and procedures for bridge inspection. The routine inspection report is most often used by bridge maintenance crews in their work. The bridge inspection report typically contains inspection commentary, photos, work recommendations and over 100 inventory, condition, and appraisal items. The bridge inspection report includes condition assessments reported in either NBI bridge components and general condition ratings or AASHTO bridge elements and condition states or both.

Each bridge inspection is documented with a report. The bridge inspection report can vary in format and style but generally has the following information included:

- Bridge identification and location
- Physical dimensions such as total length, span lengths, width, and clearances
- Information pertaining to the construction material and design type of the bridge
- Roadway functional classification and average daily traffic
- Safe load capacities and any load restrictions
- Channel condition and profiles
- Condition descriptions and condition ratings
- Inspection notes, sketches, and photographs
- Work recommendations
- Critical inspection findings

The inspection report is the origin of many bridge maintenance activities. An understanding of the report is essential for proper bridge maintenance.

The routine inspection is a visual inspection that is often done by inspectors on the ground and on the bridge deck. Special inspection access equipment is used as needed. The inspector is looking for damage, deterioration, or changes to the bridge since the last inspection. In many ways, these inspections are like routine physicals performed by a doctor. If the inspector observes anything that warrants further attention, follow up tests or investigation may be required. The routine bridge inspection report describes the conditions and defects observed by the inspector. The inspection report uses commentary, photographs, and condition ratings to document damage, defects or deterioration found during the inspection. The inspector may make recommendations for repairs or further analysis when necessary. Commentary and photographs in the report support the inspection findings and work recommendations.

Inspectors note the types of defect, locations, sizes, and causes of defect. The convention used to describe defect locations on the bridge can vary from one agency to the next. This convention must be understood to locate defects identified during a bridge inspection. The following is an example of conventions used to locate specific defects during an inspection:

- Spans are numbered from South to North or West to East
- Girders are numbered from left to right facing in the direction of the route
- Bays and girders are counted from left to right facing ahead on the route
- Substructure supports are numbered from South to North or West to East
- Lane numbers are assigned from left to right facing ahead on the route
- Rails are reported as left or right looking ahead on the route
- Joints are counted from the South or West abutment

Using these conventions, an inspector may note the location of a steel weld crack as follows:

“A four-inch crack was found in the span 4, girder #3, cross frame #4 stiffener to web weld approximately one inch above the top of the bottom flange.” Translated, this means the crack is in span #4, in the third girder from the left at the 4^{th} cross frame in the span, and is located in the weld that connects the web stiffener to the web of the girder just above the bottom flange.

This convention is an example and may not be identical to the conventions used by all agencies. It is important to learn the nomenclature convention of your agency so that defects described in the inspection report can be located. The inspector will also record a number of general condition ratings that provide considerable information on the condition of the bridge components. The ratings determined by the inspector are based on codes established in the FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, also known as the Coding Guide. The inspectors may also record bridge element condition states based on AASHTO manuals and other requirements of the owner agency. An example of a bridge inspection report is shown in Appendix B – References.

Other items from inspection reports that may be useful in bridge maintenance are listed below:

- Feature Intersected. The feature intersected item is what the bridge spans over. Features intersected include rivers, creeks, lakes, canyons, gullies, dry washes or other roads. For a bridge spanning over the Mississippi River the feature intersected would be reported as the Mississippi River. This item can be used in conjunction with the facility carried to locate a bridge.
- Facility Carried. The facility carried by a bridge is typically a roadway, but may also be a pedestrian facility, railroad, or utility line. For roadways, route number and milepost are reported.
- Structure Number. The structure number is a unique identification number assigned to the bridge. In most agencies the structure number is the primary identifier for all bridges.
- Location. The location field is a narrative description of where a bridge is located. For example; "6 miles Southwest of Richmond" on the roadway identified as the facility carried.
- Maintenance responsibility. The maintenance responsibility identifies the type of agency that is responsible for the maintenance of a bridge. This item is important because many bridges are owned by one agency and maintained by another under agreement. Before any maintenance is executed on a bridge, always verify that maintenance is the responsibility of your agency. Most agencies will keep maintenance agreement records for their bridges that provide additional detail on the responsibilities of a shared maintenance bridge. The most frequently used codes are; 01 – State Highway Agency, 02 - County Highway Agency, 04 – City or Municipal Highway Agency and 27 – Railroad. In total 23 different agencies are defined in the NBI. Refer to the Coding Guide for a complete listing.
- Open, Posted or Closed. This item indicates any operational restriction of the bridge. Typical operational restrictions include load, speed or lane restrictions. NBI codes of B, K, P or R indicate the need for posting signs (including closure) at both approaches to the bridge.
- Table 2.1. The inspector assigns a rating of 0 (low) to 9 (high) or N (no channel under the bridge). This NBI item can be used to evaluate channels that may need maintenance

work on channel or embankment protection devices such as riprap, gabions and spur dikes. Most channel maintenance activities will occur when channel ratings are between 3 and 7.

Table 2.1 Channel and Channel Protection Rating

Channel Rating	Generalized Condition
8,9	Good Condition
6,7	Fair Condition, Minor damage or bank slumping, Minor amounts of debris in channel
5	Poor Condition, Major damage or restriction, Trees or brush restrict channel
1,2,3,4	Severe Condition or closed to traffic
N	No channel under the bridge

- Operating Rating. The operating rating for a bridge is the maximum load (tons) that a bridge can safely carry. This rating is based on a truck and trailer combination with specific axle weights and spacing as defined by the AASHTO HL-93 truck (an example of which is shown in Figure 2.1 below, where 1 kip equals 1 thousand pounds). Note that the HL-93 truck is not a new design, but is the old HS20-44 design with added lane load. Higher numbers correspond with greater load capacity of the bridge. Consult a qualified engineer before making decisions based solely on this rating.

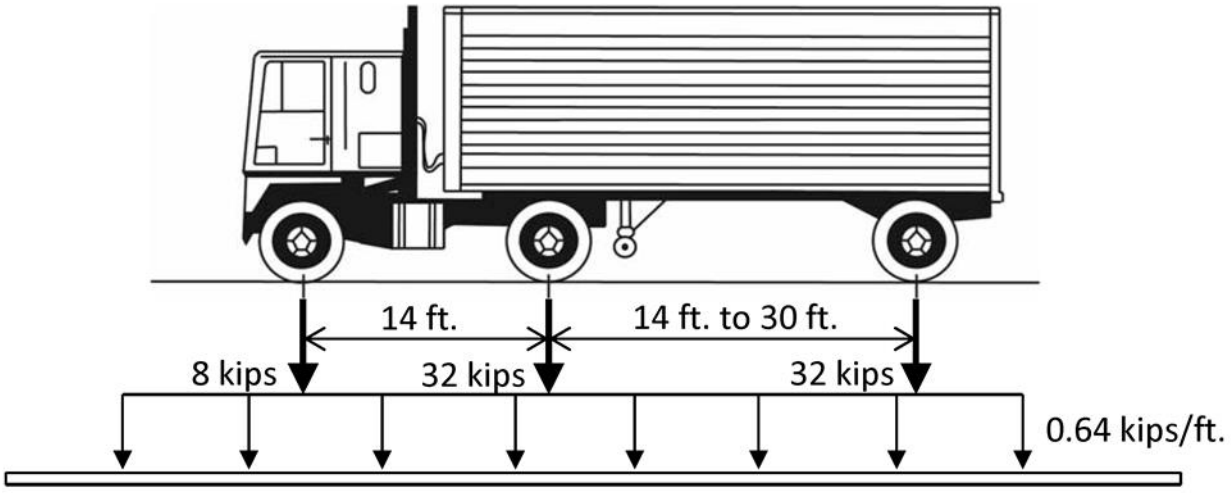


Figure 2.1 AASHTO HL-93 Truck Loading

- Scour Critical Bridges. The scour vulnerability of a bridge is reported in a bridge inspection report. Scour is the erosion of material at the bridge foundation by flowing water. Scour potential may be based on field conditions or analysis. Bridge maintenance crews should be familiar with all bridges that have scour critical codes of 4 or lower and U (where U is unknown), because these bridges require close monitoring during and immediately after periods of high stream flows. The maintenance crews should be attentive to any change in the line or grade of the

bridge railings and deck, buildup of debris at substructure elements and lateral migration of the channel. All bridges with NBI codes of 3, 2, 1 or U are required to have written scour "Plans of Action" to define specific actions that are required to ensure the safety of the public during periods of high stream flows.

2.1.2.2 National Bridge Inspection General Condition Ratings

NBI general condition ratings are assessed for the Deck, Superstructure, Substructure, and Culvert. These items are all assessed based on a 0 (low) to 9 (high) rating system defined in *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, Report Number FHWA-PD-96-0001. The reported ratings represent the overall condition of the component. Localized defects have varying impact on these ratings depending on the condition of the remainder of the component. Table 2.2 can be used an aid in understanding the NBI condition ratings.

Table 2.2 Generalized NBI Condition Ratings

NBI Rating	Description	Commonly Employed Feasible Actions
9	Excellent Condition	Preventive Maintenance
8	Very Good Condition	Preventive Maintenance
7	Good Condition	Preventive Maintenance
6	Satisfactory Condition	Preventive Maintenance/ Repairs
5	Fair Condition	Preventive Maintenance/ Repairs/ Rehabilitation
4	Poor Condition	Rehabilitation or Replacement
3	Serious	Rehabilitation or Replacement
2	Critical	Rehabilitation or Replacement
1	Immediate Failure	Rehabilitation or Replacement
0	Closed	Rehabilitation or Replacement

2.1.2.3 Element Inspection Condition Ratings

Element-level condition states and quantities are often included in the bridge inspection report. Element-level data include total quantity of each element on the bridge and a breakdown of the quantity of the each element that exists in one of four condition states. The units for quantities are EACH, FEET, and SQUARE FEET. Element numbers and their measurement units are provided in Table 2.3 and Table 2.4. See the *AASHTO Manual for Bridge Element Inspection* for a full listing of the elements and their descriptions.

Table 2.3 Frequently Used AASHTO Bridge Elements

Element	Units	Reinforced Concrete Element Numbers	Prestressed Concrete Element Numbers	Steel / Metal Elements	Timber Elements
Decks	Square Feet	12	13	28 (open)	31
Slabs	Square Feet	38	-	-	54
Rails	Feet	331	-	330	332
Box Girders	Feet	105	104	102	-
Open Girders	Feet	110	109	107	111
Truss	Feet	-	-	120	135
Stringers	Feet	116	115	113	117
Floor Beams	Feet	155	154	152	156
Columns	Each	205	204	202	206
Pier Caps	Feet	234	233	231	235
Abutments	Feet	215	-	219	216
Culverts	Feet	241	245	240	242
Pier Walls	Feet	210	-	-	212
Piles	Each	227	226	225	228

Other AASHTO elements are shown below in Table 2.4.

Table 2.4 Additional Common AASHTO Elements

Element Number	Units	Description
300	Feet	Strip seal expansion joint
301	Feet	Pourable joint seal
302	Feet	Compression joint seal
303	Feet	Assembly joint seal
305	Feet	Assembly joint without seal (Finger or similar)
310 - 316	Each	Bearings
320	Square Feet	Prestressed concrete approach slab
321	Square Feet	Reinforced concrete approach slab
510	Square Feet	Wearing surface
515	Square Feet	Steel protective coating (paint, galvanizing, etc.)
520	Square Feet	Deck protective systems (rebar coatings, cathodic protection
521	Square Feet	Concrete protective coating (Sealers, water proofing, etc.)

Element inspection condition assessments differ from the NBI component condition data in two significant ways.

- The inspection is conducted for each element independent of all others.
- The total quantity of the element is defined, and the portion of the total quantity that exists in each of four defined condition states is reported.

Every element has four condition states that have specific defect criteria as defined in the AASHTO Bridge Element Inspection Manual. Although the defect language that defines each of the four condition states varies by material and element, the following generalized condition state descriptions shown in Table 2.5 are valid:

Table 2.5 General Element Inspection Condition Descriptions

Condition State	Description	Commonly Employed Feasible Actions
1	Good	Preventive Maintenance
2	Fair	Preventive Maintenance or Repairs
3	Poor	Rehabilitation
4	Severe	Rehabilitation/Replacement

Elements can have portions of total quantity that exist in one or all four of the defined condition states.

In Table 2.6, the one square foot spall with exposed rebar is in condition state 3 because of the exposed rebar. The remainder of the 100 square feet deck element is in good condition, so 99 square feet of the element are in condition state 1.

Table 2.6 Deck Element Inspection Example

Element # - Description	Total Quantity	State 1 Quantity	State 2 Quantity	State 3 Quantity	State 4 Quantity
12 - Reinforced Concrete Deck	100 Square Feet	99 Square Feet	0	1 Square Foot	0

Now consider an example of a complete bridge, shown in Table 2.7. The element presentation is for a 50 foot long two-span reinforced concrete slab bridge with a reinforced concrete pier wall and abutments on 5 steel piles each. This bridge is 25 feet wide and has reinforced concrete rails and pourable joints at each end. The bridge is in good condition except for the failed joint seal at abutment 1.

Table 2.7 Example of a Bridge Element Model

Element	Total Quantity	State 1	State 2	State 3	State 4
38 - Reinforced Concrete Slab	50 feet x 25 feet = 1,250 square feet	1,250			
210 – Reinforced Concrete Pier Wall	1 wall x 25 feet = 25 feet	25			
215 – Reinforced Concrete Abutment	2 Abuts x 25 feet = 50 feet	50			
225 – Steel Piles	3 Supports x 5 piles = 15 each	15			
301 – Pourable Joint Seal	2 ends x 25 feet = 50 feet	25			25
331 - Reinforced Concrete Rails	2 sides x 50 feet = 100 feet	100			

2.1.2.4 Comparison of Condition Ratings

To illustrate the differences between the two bridge condition reporting systems, let's look at an example of what a condition assessment for a single bridge looks like using each system.

In this example, we have a 100 foot long, three-span, steel five-girder bridge supported on elastomeric bearings sitting on two reinforced concrete pier walls and two reinforced concrete abutments. Each substructure element is on six steel H-piles. The bridge has a 30 foot wide reinforced concrete bridge deck with an asphalt overlay that has 1/8 inch wide cracking. We have a two square foot spall in the deck with exposed rebar. The railings are steel, and all joints are compression type. The condition assessment found that all the paint on the girders has dull chalking, and a five foot length of each girder on either side of the pier joints has minor section loss from water leaking through the failed joints above. All other elements are in good condition.

Let's first consider the road surface components presented in Table 2.8:

Table 2.8 Element Inspection Condition Ratings for Road Surface Component Example

Element	Total Quantity	State 1	State 2	State 3	State 4
Reinforced Concrete Deck	100 feet x 30 feet = 3,000 square feet	2,998		2	
Wearing Surface	100 feet x 30 feet = 3,000 square feet			2,998	2
Compression Joint Seal	4 supports x 30 feet = 120 feet	60			60
Metal Rails	2 sides x 100 feet = 200 feet	200			

Using the NBI condition ratings, the inspector is to report the condition rating for the overall condition of the deck ignoring the condition of the wearing surface, joints and rails. Using the NBI condition rating system, the only defect to be considered in this example is the 2 square foot spall. The overall rating of the deck would be rated 7 (with a 9 being excellent condition).

Now let's look at the superstructure ratings (see Table 2.9):

Table 2.9 Element Inspection Superstructure Condition Ratings

Element	Total Quantity	State 1	State 2	State 3	State 4
Steel Girder	100 feet x 5 lines = 500 feet	400		100	
Protective Coating	500 feet x 8 square feet per foot = 4,000 square feet		3,200		800

Using the NBI protocol, the inspector is to report the condition rating for the overall condition of the superstructure ignoring the condition of the paint system. Using the NBI condition rating system, the only defect to be considered in this example is the minor section loss in the girders. The overall condition of the superstructure would be rated a 5 or 6.

A similar comparison of the substructure would further illustrate that the two systems capture condition in very different ways. The AASHTO element inspection is more detailed and includes condition states for joints, rails, wearing surfaces and paint systems that are not captured in the NBI component ratings. The example shown was not exhaustive, and limited to comparing the deck and superstructure for the purposes of illustrating the differences in the two rating systems

2.2 Management Systems

Many State DOTs use computerized systems to manage their bridges, pavements and maintenance programs; each of these three management systems is supported by an inspection program. In some agencies, the bridge management system (BMS) and pavement management system (PMS) interface directly with the maintenance management system (MMS).

Maintenance and repair needs of bridges are managed partly in the BMS and partly in the MMS. The following sections provide an overview of these two systems.

2.2.1 Bridge Management Systems (BMS)

2.2.1.1 Introduction to BMS

In Section 2.1, we described the types of information captured during a bridge inspection. A typical bridge inspection will generate well over 100 items of inspection related data. The bridge inspection data include: inventory information, condition assessments, inspection commentary, work recommendations and photographs and sketches. A small agency responsible for 100 bridges needs to manage over 10,000 pieces of bridge inspection data. To aid in the management of this information, many DOTs use bridge management software specifically designed for this purpose. Bridge management system (BMS) software applications are commercially available or can be custom developed to meet the needs of an agency. AASHTO develops and maintains a bridge management system software program called AASHTOWARE "Bridge Management" (formerly Pontis) that is used by the majority of state DOTs in the United States. BMS software applications have a number of features beyond just the collection and storage of inspection information. The following listing highlights some of the more common BMS features:

- Bridge inspection data storage
- Work recommendation storage
- Photographs and document storage
- Report generation
- Data mapping features
- Work prioritization calculations and rankings
- Project tracking information
- Deterioration modeling
- Definition of bridge actions and costs
- Economic decision making analysis and tools
- Bridge funding program information
- Performance measure calculations

These features support the bridge inspection program, bridge project tracking, economic analysis of potential bridge actions and performance reporting, as shown in the flow chart in Figure 2.2:

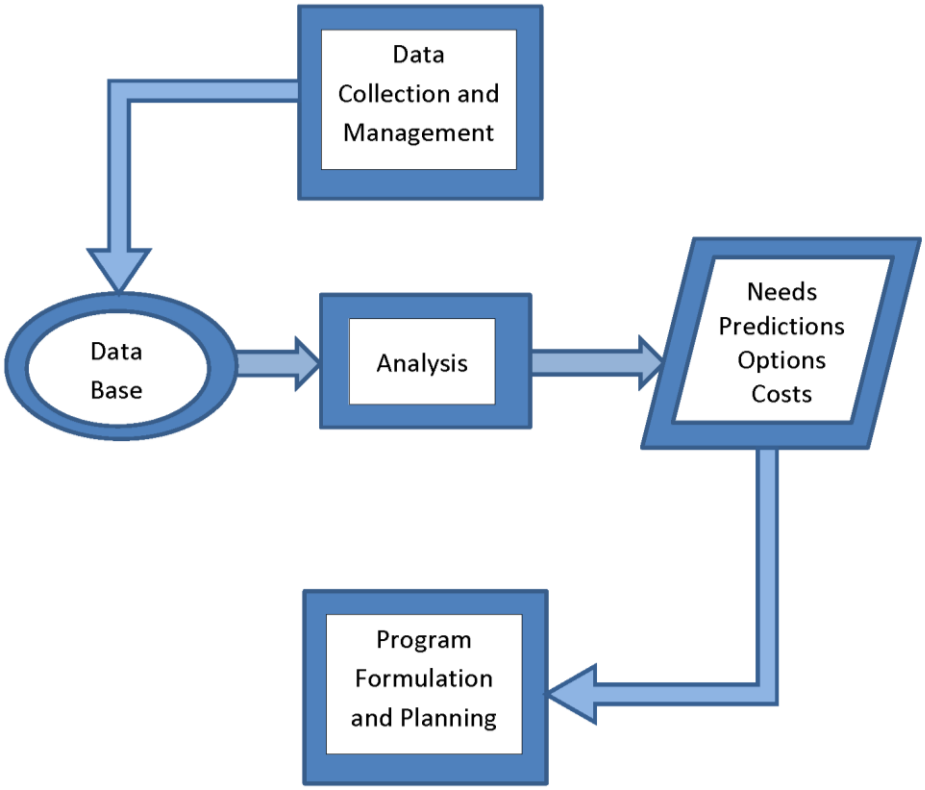
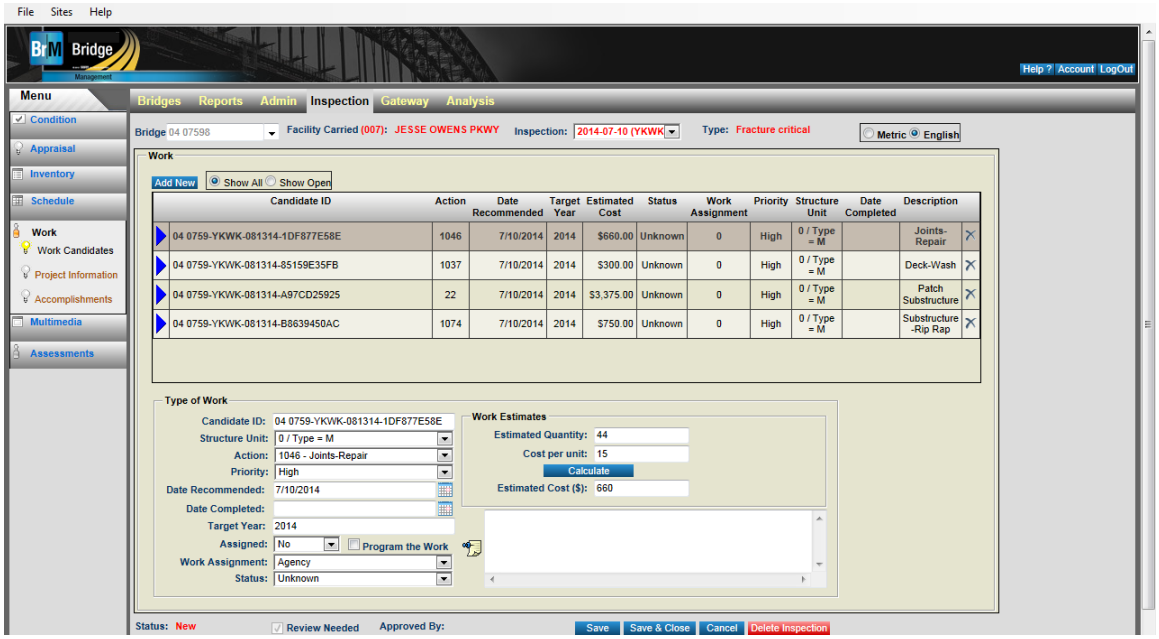


Figure 2.2 Bridge Management System Flow Chart

Bridge management systems support the inspection programs by providing a series of software screens that guide the inspector in the collection of required data. The presentation of the bridge inspection information varies among BMSs, but all should conform to federal bridge inspection requirements. Screen captures presented in Figure 2.3 and Figure 2.4 show examples of BMS bridge inspection software screens.


File Sites Help
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Help ? Account LogOut
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Condition
Appraisal
Inventory
Schedule
Work
Work Candidates
Project Information
Accomplishments
Multimedia
Assessments
Bridge 04 07598
Facility Carried (007): JESSE OWENS PKWY
Inspection: 2014-07-10 (YKWK
Type: Fracture critical
Metric English
Work
Add New
Show All Show Open
Candidate ID | Action | Date Recommended | Target Year | Estimated Cost | Status | Work Assignment | Priority | Structure Unit | Date Completed | Description
04 0759-YKWK-081314-1DF877E58E | 1046 | 7/10/2014 | 2014 | $660.00 | Unknown | 0 | High | 0 / Type = M | | Joints-Repair
04 0759-YKWK-081314-85159E35FB | 1037 | 7/10/2014 | 2014 | $300.00 | Unknown | 0 | High | 0 / Type = M | | Deck-Wash
04 0759-YKWK-081314-A97CD25925 | 22 | 7/10/2014 | 2014 | $3,375.00 | Unknown | 0 | High | 0 / Type = M | | Patch Substructure
04 0759-YKWK-081314-B8639450AC | 1074 | 7/10/2014 | 2014 | $750.00 | Unknown | 0 | High | 0 / Type = M | | Substructure -Rip Rap
Type of Work
Candidate ID: 04 0759-YKWK-081314-1DF877E58E
Structure Unit: 0 / Type = M
Action: 1046 - Joints-Repair
Priority: High
Date Recommended: 7/10/2014
Date Completed:
Target Year: 2014
Assigned: No
Program the Work
Work Assignment: Agency
Status: Unknown
Work Estimates
Estimated Quantity: 44
Cost per unit: 15
Calculate
Estimated Cost ($): 660
Status: New
Review Needed
Approved By:
Save Save & Close Cancel Delete Inspection

Figure 2.3 Screen Capture Example 1 of BMS Software (Courtesy of AASHTO)

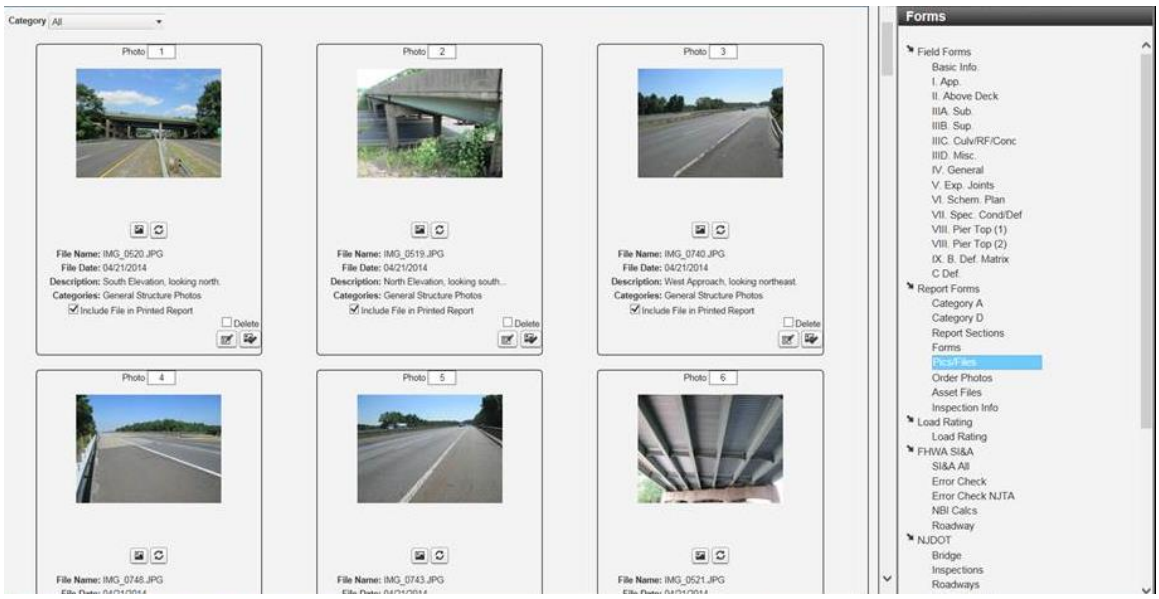

Figure 2.4 Screen Capture Example 2 of BMS Software - Courtesy of Bentley (InspectTech)

The bridge inspection and inventory data defines the bridge, its current condition and any inspector-identified work recommendations. The BMS is designed to generate many standard reports. The following reports can be generated by most BMSs:

- Bridge inspection report
- Various condition breakdowns
- Inventory listings
- Distribution of bridge age
- Material and design type breakdowns
- Ad-hoc listings of bridges meeting specified criteria
- Network element quantities
- Performance measure reports

2.2.1.2 Tracking, Identifying and Prioritizing Actions

The BMS typically stores and tracks bridge project information. The project information captured often includes the following items:

- Project description
- Project cost
- Funding sources
- Project schedule and milestones
- Bridges included in the project
- Inspector work recommendations included from each bridge in the project
- Benefit-cost evaluation for the project

Most agencies managing bridges find that the needs often exceed the funds available to fix the bridges. In these cases, the agencies must prioritize needs to find cost effective activities to

undertake in a funding year. The process of analysing existing and future bridge conditions to determine the most cost effective actions is called "optimization." In most BMSs, the optimization seeks to maximize a measured benefit relative to the cost of achieving the benefit. This is a benefit-cost ratio.

PROJECT BENEFITS / PROJECT COSTS = Benefit-cost ratio or B/C ratio

The benefit of taking a particular action is a function of the current condition of the bridge and the effectiveness of the action. For example, if we were considering repainting a bridge girder, we would look at the current condition of the paint to evaluate if there would be any benefit in repainting. If the paint was exhibiting isolated areas of distress, the benefit of spot painting to repair the distressed areas would be slightly less than repainting, but at a significantly lower cost. On the other hand, if the paint was allowed to fail, then the benefit of repainting would be significant but the cost of total removal of the paint system, repair of corrosion, and completely repainting would also be significant. The benefits of the spot painting option would be slightly less than the benefits of a new paint system, but the costs, on the other hand would be significantly different. Since the cost of preparing and painting the failed system would be much higher than painting over a paint system that has only minor deterioration, the resulting benefit-cost ratio of spot painting the paint system with minor deterioration would be significantly higher. In general, an economic optimization seeks to minimize cost while maintaining some established condition threshold or maximizing benefit for a defined budget. A BMS may perform tens of thousands of benefit-cost calculations to select cost effective projects to be funded in a single year.

BMSs can perform benefit / cost calculations in the current year, and BMSs can use deterioration models to predict the condition of bridge elements in future years. Deterioration models in BMSs estimate the number of years it will take for an element to deteriorate from one condition state to the next. Element environments are also defined so that BMSs will apply an appropriate deterioration model.

Recent advances in bridge management systems incorporate bridge vulnerabilities, such as scour and seismic risks, into prioritization. Condition deterioration and risk are combined using utility functions. In a model that uses utility theory, the benefit of an action is measured in utility gain. By comparing utility benefit to cost ratios, state-of-the-art bridge management systems can optimize all bridge needs.

2.2.2 Maintenance Management Systems (MMS)

The purpose of a maintenance management system (MMS) is to plan, program, identify required resources, schedule, monitor, document, and support maintenance activities. Maintenance management systems cover many assets found on the highway system. Assets found in the MMS vary from agency to agency but often include the following:

- Drainage features (e.g., culverts, curb and gutter)
- Roadside features (e.g., fences, brush, mowing)
- Pavement features (e.g., markings)
- Structures (e.g., bridges, overhead sign structures)
- Traffic items (e.g., signs, signals)
- Lighting

- Guardrail and attenuators

Maintenance management systems take the needs identified through inspection activities and other sources and utilize information in the system to help assign to a crew, develop a work plan, assign staff, schedule work activities, track material usage, and manage equipment.

As stated in Section 2.2, maintenance and repair needs of bridges are managed partly in the BMS and partly in the MMS. Both systems typically have an inventory of the bridges, but the BMS will have significantly more information required for the NBIS. Generally, bridge work requiring engineering design is selected, optimized and tracked in the BMS, while work requiring no engineering design is scheduled and tracked in the MMS. Reporting of completed work in the MMS is also required to be reflected in the BMS for proper management.

All non-contract maintenance needs, regardless of origin or asset, must be assigned to a crew that is going to carry out the activity. Priorities for crews are dynamic and are influenced by accidents or urgent safety repair needs. For routine bridge maintenance work, the crew supervisor must take the need identified by the bridge inspector and transform it into a specific work activity for the crew. This transformation from raw need to specific work activity involves a number of steps.

2.2.2.1 Planning

Planning may be the most important function of a bridge maintenance supervisor. The MMS helps in maintenance planning. MMSs help to achieve:

- Higher production rates on projects
- Limited “down time” in-between projects
- More efficient use of “down time” for training and yard clean up
- Improvement of the condition of the bridge inventory

Planning is the prioritization of specific bridges and maintenance activities to be completed. Of all the management functions, planning is the one function that has the greatest effect on use of labor, equipment, and materials. Supervisors should consider the whole bridge system and the bridge maintenance needs over an extended period of time. Planning begins by analyzing bridge needs and the actions and resources required for the network. Once the work plan is developed and budgeted, the plan is the basis from which specific work orders are developed, scheduled, and accomplished.

2.2.2.2 Budgeting

Budgeting is the process to quantify the funds and resources needed for work plans. Maintenance activities are budgeted by aggregate categories rather than specific work activities because the activities are each relatively small. Common budgeting categories include:

- Employee costs
- Equipment costs
- Material costs
- Training costs
- Contract costs
- Engineering costs

Cost estimates are prepared when developing budgets. The work plans developed in the planning process indicate the magnitude of work required. Past expenditures and staffing are often used to estimate budgets. It is common to have maintenance needs with costs that exceed the available budget, which is why priorities among needs are determined.

2.2.2.3 Scheduling

Scheduling is the process of laying out future work. A good scheduling process develops work schedules on at least three levels:

- Organizational level
- Supervisory level
- Crew level

Scheduling is a tool to help achieve the project objectives without exceeding the budget.

Scheduling accounts for all resources: labor, equipment, and material. Schedules are more detailed at the crew level than at the organizational level. The worker's schedules are the final approved and budgeted work plans. The crew leaders schedules are very specific as to the location, date, time, and crew members assigned to a repair. Since emergency work is always a possibility, schedules must also be flexible. Emergency conditions should be met without undue strain on the organization.

2.3 Bridge Maintenance Planning

The key to the successful execution of any bridge maintenance project is the proper planning and scheduling of the work. A relatively brief amount of time spent doing quality planning will have a positive impact on performance and will create overall time savings The planning and scheduling must be done prior to beginning the project and continue during the execution.

2.3.1 Long-Term Planning

2.3.1.1 Introduction to Long-Term Planning

Long-term planning for bridge maintenance is typically 3 to 5 years, but could be longer.

The dynamic nature of bridge maintenance needs requires that longer term plans remain flexible to adapt to unidentified urgent needs that may require the crew's attention. The continual identification of new bridge needs often necessitates that longer term planning be based on historical expenditure information to determine future budget and staffing needs.

Long-term planning involves the following steps:

- Needs are identified for all the bridges that are included.
- Preliminary cost estimates are developed, and a budget is prepared. After the budget is approved, a more detailed estimate is developed to determine the manpower and resource requirements to accomplish the work, and this is compared to the available manpower and resources.
- A decision is made to contract out the work or assign it to in house crews. This decision may be based on available staff and workload or on the need for special equipment or skills.

Priorities for work drive long-term schedules:

- The urgency of the work
- The year the work can best be performed
- When approvals or permits can be obtained
- When necessary support, equipment, and materials are available

Realistic scheduling improves efficiency. Workers should be challenged. However, if too little time is allotted to a job, shortcuts and omissions may undermine the quality of the repair work. The long-term planning effort provides general guidance regarding what to do, when to do it, and what it will take to get it done.

Once the long term planning has identified a set of work activities to be performed by the bridge maintenance crews, more detailed planning and scheduling can begin. The details of what exactly is to be done, who will do it, and what equipment and materials it will take to get the work done are included in a work order.

2.3.1.2 Work Orders

Work orders are prepared in advance and provide details of each job. The work order contains a description of the particular type of work to be performed, the exact location of the work, the names of all workers assigned to the project, and the list of equipment and materials needed to do the work. The work order may be generated in response to needs identified in a bridge inspection report, identified during other maintenance activities or through public request. At each level, long term planning, work orders, and job planning, the plan for execution of the project becomes more detailed and refined, in order to develop a plan that can be as realistic as possible about the time and resources required. The long term planning level would determine order of magnitude budget and staff requirements, Work order level is more specific in determining workers, equipment and materials, where job planning would fill in the details of how the work is actually being performed.

2.3.1.3 Job Planning

The crew leader (or a bridge maintenance engineer) usually prepares a job plan after the job is assigned to the crew. The crew leader visits the bridge and determines detailed work requirements. Factors that must be considered include:

- Traffic control and available closure windows
- Worker safety
- Environmental considerations
- How the work will be performed
- Equipment and material requirements
- Specific job assignments for crew members

A job plan is needed for every project to prevent delays.
Performance standards, if available, can be compared to the work needed. If performance standards are applicable, standards can be used to help determine the resource requirements of the job with modifications if necessary.

Performance standards define the typical time it takes for someone to perform a particular task in accordance with the agencies defined procedure. For example, a single bridge painter might be expected to prepare and paint 10,000 square feet of steel per year. If 50,000 square feet of steel needed painting, the number of crew members required to get the job done in one year could be determined (50,000 square feet/10,000 square feet per crew member = 5 crew members).

If performance standards are not available, a written description of the work to be performed should be prepared. A simple outline listing the items and order of work is sufficient. This outline of work items can be used to determine labor, material, equipment needs and to determine the time required to accomplish the repair.

2.3.1.4 Traffic Control

Any project that will place workers or equipment on or adjacent to the roadway requires traffic control in conformance with standard uniform practices. This includes correct use and placement of signs, flaggers, and delineation devices such as cones, barrels, or barricades. Most agencies have standard traffic control plans for typical situations such as shoulder and lane closures. More complex work locations may require site specific traffic control plans. The Manual for Uniform Traffic Control Devices (MUTCD) is a free publication from the Federal Highway Administration that shows proper traffic control procedures. The MUTCD contains closure plans, recommended signage and other controls for street, highway, bicycle and pedestrian traffic control.

Working on or near the highway always involves risks of vehicular accidents. Adequate traffic control is important for the crew's protection. Neglect of traffic control puts workers in harm's way and exposes the agency to very costly liability judgments.

2.3.1.5 Environmental Considerations. Permits.

Proper planning should provide adequate time to secure any necessary permits and plan for worker protection and disposal of hazardous materials. The bridge maintenance crew leader should consult with available environmental units in their organization before undertaking maintenance activities that involve asbestos, lead-based paints, toxic chemicals/materials, significant excavations or activities that require employees or equipment to be in waterways.

2.3.1.6 Worker Safety

Worker safety begins with a well thought out plan that includes ways to eliminate or mitigate hazards, proper personal protection, and properly functioning tools and equipment. Safety planning should include fall protection and confined space spans. Plans for exposure to toxic materials and for shielding against embankment collapse may be needed. Safety is practice that must be developed and reinforced on each job.

Agencies have comprehensive safety manuals. An example of the bridge inspection safety manual for New York State can be reviewed at https://www.dot.ny.gov/divisions/engineering/structures/repository/manuals/NYSDOT_Br_Insp_Safety_Manual.pdf. Daily safety meeting or pre-job safety meetings provide a reminder of the safety aspects of the job that may be encountered during the coming shift or job.

2.3.1.7 Work Procedures

The crew leader should visit the site and determine what work must be performed in order to develop the work plan. If a significant amount of time elapses between the work plan development and the work being performed, an additional visit to the bridge, closer to when the work is being performed, may be necessary to determine if changed conditions require a revision to the work plan. The magnitude of the work can change quickly. This can be especially true for bridge deck and joint work.

While at the site, the crew leader should determine the exact location and total units of work. Staging locations and storage of equipment and materials is also a consideration.

2.3.1.8 Job Assignments for Personnel

Performance standards aid in determining the required number and skills of the personnel that will be required for the task. If performance standards are not available, the required skills for each item of work in the outline of work previously prepared should be noted. The knowledge of the actual work that is to be done and the conditions at the work site may cause adjustments to the requirements. Variations may be required due to traffic considerations, unusual complexity of the job, material set or cure times and weather factors. The job needs should then be compared with the capabilities of the crew members and assignments made to appropriate individuals.

Individuals should be assigned specific tasks in accordance with their capabilities and the requirements of the job. Specific precautions or instructions should be given to the personnel at the time of the assignment. If required skills are not available among the crew, necessary personnel from other DOT units or contractors should be obtained.

2.3.1.9 Equipment Requirements

The performance standards aid in determining the type and amount of equipment required to properly perform the work. Modifications may be necessary due to unusual work requirements, personnel qualifications, or differences in the equipment available compared to equipment identified in the standards. If performance standards are not available, units of equipment required for items of work in the previously prepared outline should be noted. Back-up units may be necessary or alternative plans made so equipment failures will not delay completion, especially for critical tasks or when a reopening to traffic is mandatory.

Tools required for a task must be considered and compared with the equipment and tools available to the crews. If additional tools are required, these must be procured and brought to the site to avoid delays. Every crew should carry a set of standard tools and know the procedures to obtain back-ups in the cases for breakage or failure during the job.

2.3.1.10 Material Requirements

The amount of materials needed for the job can also be determined from the performance standards or estimated from past experience. Any materials requiring a special order should be obtained before the work begins. A reserve of the materials should be taken to the job site in case quantities were underestimated. Other materials that were not identified in the work order that might be needed, such as wood for forming, cribbing, or blocking, Steel plates and

shims, concrete anchors, bolts, joint backing material, back filling material, concrete patching material, clean up items, etc. should also be included.

2.3.2 Scheduling

Once the job plan has been prepared, the work can be scheduled. Managers develop monthly work schedules and refine the schedule weekly. When scheduling work for the month, it is important that the maintenance activities be coordinated. Coordination includes performing similar work on multiple sites on the same day or in the same week to increase efficiency. Examples of good scheduling include repairing concrete on several bridges in the same area at the same time or cleaning and painting several adjacent bridges at the same time. Bridge maintenance should be coordinated with DOT units for traffic operations, public affairs, and with other construction and maintenance operations on the roadways.

At the end of each week, the schedule for the following week is developed. During this process the following steps should be taken:

- Check to see if any work scheduled for the present week will be carried over into the next week and possibly delay planned work.
- Review the work scheduling to see which new activities are to receive the highest priority.
- Review the monthly schedule to see what projects are scheduled and if adjustments are necessary.
- Determine the employee days, equipment, and material required for the week. If manpower is still available, additional fill-in projects should be identified.
- Complete the schedule by assigning crew members and equipment to the specific projects.

Inclement weather may disrupt the work schedule. It is important that other work is planned that can be performed in bad weather so that crew time will not be wasted. In many states, repairs are generally limited in the winter months. Since winter work performed is minimal, this period is ideal for the training of employees and the repair of equipment. Many states use bridge maintenance crew members to plow snow, fabricate precast concrete bridge elements, and perform other incidental jobs in the winter months.

2.3.2.1 Critical Path Scheduling Method

The Critical Path Method (CPM) is one method of scheduling sub-tasks of a project or maintenance procedure. CPM has many applications. CPM can be used for scheduling tasks within a work order or for the yearly activities of the entire maintenance unit.

In CPM**,** each activity is represented as a task on a network, and connecting lines are drawn to represent the time schedule to complete that activity. The following steps are typical for CPM:

1. Identify the activities
2. Determine the sequence of the activities
3. Connect activities in order of sequence
4. Enter the time required to complete each activity
5. Identify the critical path or the longest possible path to complete all activities
6. Update progress as tasks are completed

2.3.2.2 Identify Individual Activities or Tasks

A complete list of activities needed to complete the project must be made. This list can be prepared from the work outline or the work plan. Sequence and duration of activities are set in subsequent steps.

2.3.2.3 Determine the Sequence of the Activities

Determine the sequence of the activities in the project. The activities are sequenced according to their dependencies on one another. Activities must be analyzed and sequenced properly to achieve an accurate result. If linked incorrectly, an incorrect time frame or erroneous project completion date may result. If one crew is being used to perform similar activities, these activities must be sequenced.

2.3.2.4 Creating the Network

When all activities have been defined and sequenced, a network of activities can be created. The network is the ordinal coordination of activities.

2.3.2.5 Estimate Activity Duration

The time necessary to complete each activity is determined using workload standards or past experience.

2.3.2.6 Identify the Critical Path

The critical path is the longest-duration path through a series of tasks necessary to complete a project**.** Activities located on the critical path cannot be delayed without delaying the overall project. The critical path method of scheduling is often presented using a diagram that shows the specific tasks, their duration and sequence. This critical path diagram is often presented in a Gantt chart. CPM scheduling with automated Gantt charting is readily available in spreadsheet form or in project management software used by most transportation agencies. These software tools can help streamline the scheduling of projects that involve many tasks.

2.3.2.7 Updating the Gantt Chart

For longer duration activities, the critical path schedule must be updated based on actual activity accomplishments. A new path might be found and alternative schedules can be presented to accelerate the project. Sometimes it is necessary to incorporate extra work that was not part of the original critical path. Training in critical path method is commercially available and recommended for bridge maintenance employees responsible for planning work activities.

Critical Path Example – Perhaps the best way to understand the critical path method is by example. Consider a project to remove an old pourable joint seal and install a new one. The project has the following tasks:

1. Acquire the new pourable joint seal material
2. Acquire round foam backing for under the new joint
3. Remove the old pourable joint seal
4. Clean the inside concrete edges of the joint
5. Place the new foam backing into the joint

6. Extrude the new joint seal material into the joint

With the tasks identified, the sequence of activities is determined. In this case, the sequence is obvious, however there are still options. For example, a two-person crew could complete Task 4 in half the time compared to one worker.

The network of activities is developed next. The network requires two workers. A duration to each task is assigned (Table 2.10):

Table 2.10 Development of Network of Activities Table

Task	Time to Complete
1. Acquire the new pourable joint seal material	2 hours – Need to go to the store
2. Acquire foam backing material	10 minutes – Have material
3. Remove old joint material	20 minutes
4. Clean concrete surfaces	120 minutes
5. Place new foam backing material	10 minutes
6. Extrude new joint seal into prepared joint	20 minutes

The tasks, sequence, and time required for each task can be transformed into a Gantt Chart, as shown in Figure 2.5:

Task	Time Required and Sequence
Task 1 – Acquire seal material	120 min.
Task 2 – Acquire foam backing	10 min.
Task 3 – Remove old joint	20 min.
Task 4 – Clean The joint	60 min.
Task 5 – Place Foam backer	10 min.
Task 6 – Extrude new joint seal	20 min.
Total Project Time	3 hours and 50 minutes

Figure 2.5 Gantt Chart for Pourable Joint Seal Replacement

The gray entries are on the critical path. Task 2 is not in the critical path because while one person is going to the store for the pourable joint seal, the other person only needs 10 minutes to get the foam backer material (20 minutes are idle). Task 3 should not be started until all required materials are at the job site. Task 6 cannot begin until all previous tasks are completed. By having two people available, Tasks 1 and 2 are able to be completed at the same time. Additionally, 60 minutes are saved by having both crew members clean a portion of the whole joint (a 120 minute task). Although this is a very simplified example, it demonstrates the power of critical path scheduling.

2.3.3 Performing the Work

Performing the work is the process of actual completion of the work in the field. The work should be done in a manner that conforms to quality and performance standards.

The crew is responsible for reviewing the work plan with the crew and for assigning job tasks. The work crew leader should plan the work in detail. This usually involves determining the specific work methods (how to do the work), equipment requirements, materials needed, and staffing the actual performance of the work. The work crew leader may also make revisions or

refinements to the work plan based on the input from the crew members who will actually be performing the work.

The crew supervisor reviews the safety plan for the job with the crew prior to the work. Safety aspects may include traffic control, personal protective equipment, heavy equipment usage, material safety, etc.

After the maintenance activity has been fully planned and scheduled, the focus of the crew shifts to executing the repair plan. The daily routine for any job consists of the following basic steps:

1. Maintenance yard housekeeping activities
2. Check of proper operation of equipment and vehicles at the yard
3. Preparation for the job at the yard and moving to the job site
4. Establishment of traffic control, if necessary
5. Performance of the work at the job site in accordance with the job plan
6. Cleanup after the job at the yard

The daily routine contributes to efficiency of performance of work. Each step has a number of subtasks that are identified below:

2.3.3.1 Housekeeping

The maintenance yard is the base of all operations. The yard must be maintained in good condition to facilitate field operations and ensure worker safety. Most states have a Safety Engineer or equivalent person that helps yard supervisors keep the yard safe in compliance with Occupational Safety and Health Administration (OSHA) requirements. Bridge maintenance supervisors should strive to develop a close working relationship with these support people, and work with them to develop a Standard Operating Procedure (SOP) for yard maintenance activities. Some items in the SOP include:

- Periodic update of material safety data sheets (MSDS)
- Periodic check of parked vehicles for fluid leaks
- Check of yard safety signs for compliance and legibility
- Check of stored materials to ensure they are safely stacked as appropriate
- Return of all flammable/combustible materials to the designated storage areas
- Periodic check of flammable/combustible material storage areas for compliance, and to ensure reactive materials are separated
- Proper storage of all compressed gas cylinders is in compliance with regulations
- Check and service of fire extinguishers as required
- Check and clean-up of all hazardous waste storage areas
- Disinfecting clean-up of all common areas (shower, toilet, shop floor, etc.) daily

Crew supervisors should maintain health and safety records for their crew members. The records should include dates of relevant training (respirator fit testing, fall protection, confined space, etc.) and medical testing (such as blood lead levels, etc.).

2.3.3.2 Maintain Equipment and Vehicles

Equipment and vehicles must be maintained in good working order. All equipment and vehicles should be checked with adequate lead time for repair and serviced prior to mobilization. Checks and services should be performed in accordance with the manufacturer's schedule and guidelines.

2.3.3.3 Yard Preparation

Prior to leaving for the job site, the crew is first assembled to determine if changes must be made in job assignments. The crew members should be informed of the tasks that each is to perform. This allows each crew member to know what the assignment is and what teams will work together. The crew then gathers the required equipment and material needed.

2.3.3.4 Site Activities

When the crew arrives at the work site, the crew leader should review the safety procedures to be followed on the job and should ensure that safety-related assignments are fully understood. Once proper traffic control has been established, the crew can proceed with the work. Traffic control cones, signs, and flags should be checked periodically and the site should be kept as clean and uncluttered as possible while the work is underway.

When the job is complete, the tools and excess material should be properly stowed prior to leaving the site. Once the site is clean and again suitable for traffic, the traffic control devices can be removed. To properly reopen the highway to traffic, signs and traffic control devices should start at the work site and proceed back from the work site in the opposite direction of traffic.

2.3.3.5 Yard Clean-up

There are tasks that must be performed after the crew returns to the yard. The tasks include:

- Storing the tools and materials in their proper locations
- Disposing of debris
- Preparing equipment for the next use
- Reporting the work accomplished

To save time the next work day, all equipment should be serviced, including adding gas and oil. Problems encountered with the equipment during the day should be reported. The final tasks for the crew lead is reporting the work accomplished during the day and ensuring that work for the following day is planned and resources for performing it are available.

2.3.3.6 Staffing

To ensure that the job can be performed properly, the crews should include personnel who have been trained to perform the tasks that are required. This includes such specialists as:

- Carpenters for form building and wood working
- Welders for steel work and certified welders for primary members
- Laborers for concrete and masonry work
- Operators for special equipment
- Paint crews

These specialists are able to perform a majority of the tasks. The remainder of the crew should be made up of general laborers who can assist the specialists and perform other required activities. In addition, the crew leader should have the necessary equipment and stockpiled material to ensure that the scheduled work can be performed.

2.3.3.7 Equipment

Standard equipment for each bridge maintenance crew should include:

- Fall protection
- Under bridge inspection vehicle for some bridges
- Flat-bed truck with winch and lifting equipment
- Pick-up trucks
- Pole trailer or unit for lengthy materials
- A variety of air-powered tools
- Air compressor
- Small concrete mixer or mortar mixer
- Oxygen and acetylene welding and cutting equipment
- Heavy duty jacks (20-ton to 50-ton hydraulic)
- Portable arc welder with electric service outlets
- Heavy-duty electric drill fitted with an electromagnet
- Small, portable high pressure water pump
- Sandblasting equipment
- Hand tools: steel, carpentry, concrete, and mechanical
- Staging (scaffolding)
- Painting and containment equipment
- Tow cable and chains
- Radio equipment
- Chain saw
- Heavy duty chain hoist
- Temporary bridge sections and parts (as needed)
- Bolt wrench appropriate for the method of high strength bolting used by the agency
- Pneumatic hammers
- Concrete Saws
- Digital camera to instantly record activities
- Miscellaneous survey equipment (tape, level-rod, etc.)

2.3.3.8 Materials

Each bridge crew should have a small supply of materials, especially for emergency repairs.

Materials can be ordered or accumulated from salvaged materials or taken from job site left overs:

- Supplies to maintain the type of expansion devices commonly used by the agency
- Timbers for blocking and cribbing
- Concrete repair products
- Bridge joint materials

- Assorted bridge planks
- Steel decking
- Assorted I-beams, angles, channels and plates
- Reinforcing steel
- Sheet piles
- Timber and steel piling
- Cement, mortar, mason sand, aggregates, stone fill
- Epoxy
- Painting and painting supplies
- Nails, spikes, bolts, nuts, washers, pins, lag screws

2.3.4 Reporting Bridge Maintenance Accomplishments

Once the bridge maintenance activity is complete, it is important to report the details of the repair operation. It is important to document the repair because future decisions are based on the information collected from each job. Information provided by workers allows managers to maintain:

- A historical record of maintenance and repair activities, quantities, and costs
- A record of regular periodic and special expenditures as a basis for developing future budgets
- A current record to establish cost/performance relationships
- A source of information used to identify trends, and the need for additional cost or work items
- A source of information for public relations, accomplishment reports, and defense of tort liability claims; and
- A record of costs versus budget

Reporting is the primary means of communication between maintenance and agency management. Work completions and the use of resources are reported as they occur. Work completions inform management that planned and scheduled maintenance goals have been met and provide data to account for funds.

Each agency has specific reporting requirements. The following is a brief description of the information requirements.

2.3.4.1 Who did the work

The crew and the individual in charge of the work should be specified. This allows the work to be charged to the proper department and allows future verifications and follow-up in case of discrepancies or claims.

2.3.4.2 Work accomplishment

Most MMSs have work order numbers associated with maintenance projects. This work order contains all the information regarding what work is to be done. The amount of work actually performed should be recorded. The performance standards name the basis for measurement. In addition, the bridge repaired should be reported by bridge identification number or by route

and milepost, or both. This information makes the work part of the repair history of the bridge. This also provides a notification back to the inspector indicating the work has been completed.

2.3.4.3 When was the work done

Report the date(s) the work was performed. This information is helpful in determining when work should be scheduled in future years and is required for scheduling periodic maintenance, particularly preventive maintenance.

2.3.4.4 Where was the work done

The location of where the work was performed on the bridge should be included so that the site of the repair may be easily determined. Information such as span number, travel direction, lane number, beam number, substructure number, sign inventory number, etc. would be used to provide exact location.

2.3.4.5 What was the effort to accomplish the work

This reports the resources used and how the job was performed. The hours of labor, types of equipment, and types and amounts of materials used are included. This permits computing the cost of performing the work and provides utilization data. The information also permits the determination of monthly resource needs for use in future scheduling.

2.3.4.6 Reporting Procedures

The actual reporting of the work performed has many variations. Most states use maintenance management system software for reporting. Personnel in the field furnish most of the information for the MMS. The individual in charge of a crew has the best knowledge of what transpired. Good and accurate reporting is obtained when the crew leader performs the close out of a work activity immediately after the work is completed. Many agencies keep before and after digital photos to show the work that was done. Completed work also needs to be reported back to the agencies bridge management system.

2.3.4.7 Evaluating

Evaluating is the measure of quantity and quality of work. Provisions should be made so that those most involved with the actual work, the crew leaders for example, regularly receive information so they can assess their own performance. Evaluations are based on comparisons between like crews or personnel performing like tasks with due account taken for extenuating factors such as individual work site specifics. Comparisons are also made against standards, so the standards may be adjusted to make expectations reasonable and so that plans and budgets are more accurate. Evaluation is also the system element that serves to measure the effectiveness of all of the other elements and to provide the means by which not only bridge maintenance, but the personnel performing it, can be improved. A maintenance management system can supply the manager with the information to more effectively and efficiently accomplish their maintenance program. At the same time the system will increase the control of operations and provide credibility to the program since the manager will have information with which to respond to requests and justify budgets.

2.4 Bridge Maintenance Contracting

2.4.1 Introduction

Most agencies use contract maintenance for some portion of bridge maintenance work. Agencies contract for bridge maintenance work to meet peak workload or to get specialized equipment or skills. This section summarizes contracting approaches used for bridge maintenance.

2.4.2 Types of Maintenance Work Contracted

Almost any type of bridge maintenance work can be provided through contracts. Agencies may contract for specific activities such as bridge painting or deck sealing where the location of the work is very specifically defined. Alternatively, agencies may opt to contract for a broad range of maintenance activities in a defined geographic region. Agencies engage contractors that are available for "on call" emergency work on bridges. Many agencies balance the bridge maintenance workload between agency crews and contractors to make effective use of resources and to provide good service to the highway users.

2.4.3 Maintenance Contracting Approaches

There are two types of contracts that are frequently used for bridge maintenance work. In the most common types of contract, the agency defines how the work will be done. This contract is a method-based contract. In the other type of contract, the agency defines the expected outcomes of the work. This contract is a performance-based contract. The two types of contracts have different levels of contractor control over the work and associated risk for the completed work. Method-based contracting gives the agency more control over the work and more risk for the end product while performance-based contracting provides less control over how the work gets done, but comes with less risk to the agency for the end product.

Table 2.11 Contract Type Risk and Control Summary

Contract Type	Agency Control	Agency Risk Level	Contractor Control	Contractor Risk Level
Method-Based	High	High	Low	Low
Performance-Based	Low	Low	High	High

These two contract types have a number of variant forms that shift the degree of control and risk between the agency and contractor. There are maintenance needs that are more suited to one contracting method or the other. The variant forms of the two basic contracting types are summarized below.

2.4.4 Method Based Contracting Methods

Historically, agencies have used method-based contracts to control the work that is being performed. These contracts are either agency directed or specification based.

2.4.4.1 Directed Contracts

Contractors bid on directed contracts by submitting unit prices for an identified schedule of bid items. The items bid would be work that is likely to be necessary during the contract period, as

well as a rough estimate of the work likely to be done. The individual bid items are summed up to determine the lowest bidder for the contract. Agencies may offer a directed contract for bid that covers a geographic region rather than specific bridges. Directed contracts typically pay the contractor by the amount of work they do.

In directed contracts, all of the risk for errors in quantity, plans, and site conditions falls on the agency, with minimal risk to the contractor. Directed contracts typically require significant agency resources to direct the contractor during their work.

Directed contracts minimize some of the financial risk to the agency because the agency has significant control over the work. In many directed contracts, the agency controls financial expenditure through the issuance of work orders for smaller portions of the total contract value. The issuance of work orders is a way to address fluctuating maintenance budgets.

2.4.4.2 Specification Based Contracts

In specification based contracts, the agency specifies how the work is to be performed. Key components of this type of contracting are plans and/or specifications that govern how the work is to be done by the contractor.

The agency risk associated with specification based contracts is high and the contractors risk level is slightly higher than in directed contracts. In specification based contracts, the contractor follows the agency defined specification; however, the means that they use to follow the specification is up to the contractor to determine.

Warranties have been used successfully on specification based contracts to ensure the desired project outcomes and length of service of the completed project. The warranties are backed by construction bonds issued to cover any necessary rework.

Specification based contracts are awarded to the lowest qualified bidder. If warranties are required by the agency, the costs of the bonds are included in the contractors bid.

The advantage of specification based contracts is that the agency retains control over the work through the specification. The control over the work comes with risk to the agency for any errors or omissions in the specification. Any changes to the specification after award can result in additional costs to the agency.

2.4.4.3 Job Order Contracting

Job order contracting is a variation of a specification based contract. A Job Order Contract (JOC) uses standard plan details for bridge maintenance and a universal specification for maintenance activities that are covered under the contract. The main characteristic of a JOC is that the quantity of work is indefinite. These contracts use a “price book” that has all the individual bridge maintenance bid items included. Unlike direct contracts, the price book has an extensive list of bid items covering virtually every maintenance task the agency performs. The price book also has no quantities estimated. The price book is the contractor’s bid price or mark-up from an indexed price for each maintenance item. The main difference between other specification based contracts is that the contractor does not know where or when the work will occur within a defined region. This contracting form allows the agency to issue work orders or “job orders” that specify specific locations where items in the price book are to be performed. The contractors have bid items for mobilization and traffic control in the JOC. This essentially allows

the agency to enter into a contract without knowing the exact amount of any specific bridge maintenance item that will be needed. JOCs are known for minimization of claims that may otherwise arise from uncertain limits of work common with bridge maintenance.

2.4.4.4 Equipment Contracts

Contracting for specialized equipment not owned by the agency is a form of specification based contracting. An agency will enter into a contract for a variety of specialized equipment that is requested by task order and/or bid items. The contract has fixed "bid" prices for certain classes of equipment on an hourly, weekly, or monthly unit price basis. When the equipment is needed, the crew supervisor issues a task order for a duration of time. In some cases, the equipment may be offered with an operator as part of the contract unit price.

2.4.5 Performance-Based Contracting

Performance-based contracting relies on agency-developed performance specifications that defines the desired outcome of the work but not the methods or means used to achieve the outcome. Performance-based contracts are used to provide all aspects of maintenance on a single bridge or highway corridor known as total contract maintenance.

In performance-based contracts, the agency must define specific level of service requirements for each of the items or assets included in the contract. The success or failure of a performance-based contract depends on the terms defined in the contract document. The performance standard must be adequately defined to result in the desired outcome.

For performance-based contracts, the contractor is responsible for determining the effort associated with achieving the agency defined level of service. This contracting approach effectively shifts the risk of errors in the determination of quantities from the agency to the contractor.

2.4.6 Comparison of the Contracting Approaches

The following example demonstrates the difference between the contracting approaches.

In this simplified example, an agency wishes to contract for bridge deck spall repair work on multiple bridges in a region.

Under the directed contract, the agency solicits bids from multiple contractors. The agency verifies that the contractors are licensed and bonded. A contractor is selected based on the lowest unit cost. The agency then issues a work order describing the specific locations and quantity of spall repair to be performed. The agency verifies the work as it is completed and pays the contractor according to quantity of work completed.

Under a specification based contract, the qualified contractors are selected based on lowest bid cost. A work order is issued for specific locations and quantities to be repaired similar to the direct method. The contractor completes the work using methods that satisfy the agency specification. The agency verifies through field inspection that the work meets the specifications. The contractor is paid based on the unit quantity of work completed according to the specifications.

For the first two methods, the agency performs its own quality assurance program to ensure compliance on the part of the contractor.

To use a performance-based contracting method, the agency must first develop a performance specification that completely defines the level of condition expected for bridges in the region and the level of deviation from this expectation that will be tolerated before a finding of non-compliance. The agency solicits bids from qualified contractors based on the performance specification. The contractors submit fixed price cost estimates to complete any spall repair work necessary to meet the performance specification over a defined period of time. With this approach, the contractors bid is based on their estimate of the current and future spall repair needs for the defined performance over the duration of the contract. The agency then monitors the performance of the contractor in keeping bridge deck spalling within the performance requirement. Payments are made to the contractor on a regular basis when the performance meets the defined level of service.

2.4.7 Common Methods of Payment

The agency has several options for how the payment to the contractor will be structured for maintenance contracts.

A common payment approach used for method-based contracting is unit payment. This payment approach can be used for directed or specification based contracts. Using this payment approach, the agency pays the contractor for the amount of work completed in accordance with bid unit prices.

If the work can be defined with a high degree of certainty, the agency may opt for a lump sum payment. Using the lump sum payment, the agency pays the contractor a single price to complete the work. The lump sum approach can be used with specification based contracts, but is not well suited for directed contracts because the quantity of work is not always defined at the time the contract is bid. Performance-based contracts are typically bid as lumps sum and paid in installments according to the contract payment schedule if performance is adequate.

If it is too difficult to define the amount and scope of work, it may be necessary to resort to a cost reimbursement contract, or force account contract. In contracts of this type, the contractor is reimbursed for their labor, equipment, and material costs at predetermined rates. Cost reimbursement contracts require a great deal of inspection and record keeping on the part of the agency to verify the contractor's charges. Another problem with cost reimbursement contracts is that there may be a perceived, if not actual, loss of competition. The contractor may be perceived to be working at a slower rate to increase labor costs, or using newly purchased material to increase their mark up on materials. These steps by contractors to make more money in a cost reimbursed contract means that the agency representatives is not only there to monitor quality in accordance with agency standards (specifications), but also to monitor productivity and measure quantities. Even if the inspector is not satisfied, it is difficult to document poor performance unless it is well below standard. The following table summarizes the payment options for common situations encountered in bridge maintenance contracting:

Table 2.12 Maintenance Contract Payment Methods

Payment Type	Directed	Specification Based	Performance-Based
Unit Price	Yes	Yes	No
Lump Sum	No	If scope is well defined	Yes
Cost Reimbursed	Yes	Yes	No

2.4.8 Methods of Contractor Selection

For all types of contract, the method used to select the contractor must also be determined. Options for contractor selection are presented in the following sections.

2.4.8.1 Lowest Responsible Bidder

A common contractor selection method is to select based on lowest responsible cost. Under this approach contractors respond to a request for bid by submitting a detailed cost estimate along with other required documents. The bid with the lowest "responsible" cost estimate is awarded the contract. A responsible bid may be evaluated using a number of methods, but often involves some analysis of cost deviations among the bidders.

2.4.8.2 Qualification and Negotiation

The qualification and negotiation approach begins by the agency reviewing the qualifications of the contractor. Those contractors that are deemed qualified then enter into negotiations with agency to determine the price for the work to be performed. This approach is common for engineering and design services. Under this selection method, the contractor is selected based on qualifications and not solely on cost.

2.4.8.3 Best Value

The best value selection approach is a hybrid of the two previous methods. Using the best value approach, technical qualifications are used to develop a short list of contractors. The short listed contractors are asked to submit a technical proposal and cost proposal in two separate packages. The review panel evaluates the technical proposals without opening any of the cost proposals. Once the technical proposals have been assigned a score based on an agency defined evaluation criteria, the cost proposals are opened and ranked independent of the technical proposal scores. The two component scores are combined into a total proposal score. The contractor with the maximum combined score is selected.

2.4.8.4 Summary

Contracting for bridge maintenance can be useful. Contracts can be directed or performance-based. Within these general categories there are a number of variations of how the costs are determined and how much flexibility the contractor has to determine how the work is performed. Likewise the various contracting options provide varying levels of control over the work and costs for the agency.

2.4.9 Comparing the Cost of In-House Crews Versus Contracting

When determining whether or not to contract, or when estimating the total cost of a project to be done by contract, the incidental costs of contracting should not be neglected when developing estimates. Incidental costs can be appreciable, and they can vary with the type of contract. Costs incidental to contracting include:

- Administrative costs of preparing requests for proposal or bid documents
- Costs of selecting the contractor or vendor *(*particularly for negotiated contracts)
- Administering the contract while the work is being done
- Inspecting the work
- Field record keeping
- Consultations with the contractor
- Verifying charges, approving final results, and approving invoices for payment

There is a tendency when comparing contract to in-house cost to compare the salaries of the agency employees with those of contractors and ignoring the hidden cost to the agency. Other costs of performing work in-house that should be considered include:

- Overhead costs such as employee benefits, work space and administrative support
- Cost of keeping employees on the payroll during bad weather, during winter months, or when the workload is light
- Cost of purchasing equipment and keeping it operational
- Liability costs associated with the maintenance operation such as injury to employees or the traveling public

2.5 Quality Control and Quality Assurance

2.5.1 Introduction

Quality control and quality assurance programs are important parts of an agency's overall bridge maintenance program. Quality assurance establishes process and procedures that will lead to high quality maintenance. Quality control programs verify that the quality standards established are being met.

2.5.2 Quality Control / Quality Assurance Definitions

The terms quality control and quality assurance are often incorrectly used interchangeably. One basic distinction between the two is in the timing of when the quality is influenced. To properly apply these concepts we need to first understand the difference between them as they apply to bridge maintenance.

2.5.2.1 Quality Assurance (QA)

Quality Assurance (QA) refers to the processes or procedures used to perform bridge maintenance. Quality assurance makes sure you are doing the right things and in the right way. Examples of quality assurance include process checklists, repair procedures and standards development.

Quality assurance activities are determined before production work begins. Quality assurance is also the compliance with those checklists, procedures, and standards as activities are

performed while the repair is underway. QA aims to prevent inferior work with a focus on the process used to perform the repair. QA is a proactive quality process.

2.5.2.2 Quality Control (QC)

Quality Control (QC) refers to activities associated with the creation of project deliverables. Quality control is used to verify that deliverables are of acceptable quality and that they are complete and correct. In other words, quality control makes sure the results of what you've done are what you expected. Examples of quality control activities include inspection and testing process.

Quality control activities are performed after the work is complete. QC aims to identify and correct defects observed or measured in the finished product. Quality control is a reactive process.

2.5.3 Quality Management Plans

A Quality Management Plan (QMP) combines the objectives, standards, activities, and measures of the quality assurance and quality control program into a single plan to address the overall quality of bridge maintenance. The QMP defines the acceptable level of quality and describes how the maintenance crews will ensure this level of quality in their deliverables and work processes. The QMP describes the following quality management components:

- Quality objectives
- Project deliverables and processes to be reviewed for satisfactory quality level
- Quality standards
- Quality control and assurance activities
- Quality roles and responsibilities
- Plan for reporting quality control and assurance problems

2.5.4 Quality Assurance in Bridge Maintenance

A good bridge maintenance program requires that the agency ensure bridge maintenance activities are being done in the proper way. Agencies that employ quality assurance typically focus their quality assurance efforts in the following areas:

- Standards development
- Work Procedures
- Employee Training

2.5.4.1 Quality Assurance Standards

The quality assurance standards are a set of desired outcomes. These standards define quality bridge maintenance repair. For example, consider the desired quality standards for a concrete deck repair activity. A quality assurance standard for this type of repair might include the following quality standards:

- The newly placed concrete shall be solid and crack free
- The spall repaired area shall be at the same finished grade as the surrounding deck
- Acute angles shall not be permitted in a spall repair
- The defined surface friction shall be present in the repaired area

The criteria presented above establishes the expected quality outcome for a concrete bridge deck spall repair activity. The crew uses defined work procedures to achieve the desired quality in the finished repair.

2.5.4.2 Work Procedures

Work procedures outline the steps necessary for the proper completion of a bridge maintenance repair. As an example, consider the repair of a concrete bridge deck spall. A work procedure for this activity might look like this:

Suggested Procedure

Concrete Deck Repairs

1. Identify the deck location to be repaired
2. Control traffic in accordance with traffic management plan
3. Hammer sound or chain area around the spall to identify and mark adjacent unsound concrete.
4. Sawcut to a depth of at least ¾ inch in a geometric pattern 1 inch outside the lines marked in the previous step.
5. Use a lightweight chipping gun to chip the concrete out between the edge of the spall and the sawcut lines. Remove the concrete to at least 1 inch below the top mat of reinforcing steel.
6. Use sand blasting equipment to remove any rust from exposed reinforcing steel.
7. Replace any missing steel with proper overlap lengths. Consult an engineer as necessary.
8. Remove any loose concrete material or other debris from inside the spalled area.
9. Wet the concrete in the spalled area and surrounding but do not allow standing water.
10. Mix rapid setting concrete and place concrete in the spalled area up to the surface of the surrounding deck.
11. Use a 2 by 4 or similar lumber to screed the newly placed concrete to the level of the surrounding deck.
12. Use a wet broom to provide a roughened surface in a transverse direction to traffic.
13. Use water and broom to clean the areas around the newly placed concrete.
14. Remove all materials and tools from the work site while the concrete is setting.
15. Allow the concrete cure to achieve adequate strength before restoring service to traffic.
16. Remove the traffic control and restore service after the concrete has set.

With a procedure similar to the example shown above, it is clear what steps are needed to end up with a good quality spall repair. Work procedures similar to this example can be created for all bridge maintenance activities. The establishment of defined work procedures is an essential tool to ensure that bridge maintenance activities are done consistently from job to job and from crew to crew. Adherence to the defined work procedures will help ensure that good quality is achieved.

Note that throughout this manual, suggested procedures will be flagged in call out boxes such as the one shown above. These procedures are only suggestions and should be confirmed by management.

2.5.4.3 Employee Training

Proper training is an important component of quality assurance for bridge maintenance activities. For example, it isn't reasonable to expect that a bridge crew member could meet the required quality standards for welding without appropriate training. Likewise, the resulting quality of a bridge painting project is likely to be below quality standards if the painters are untrained.

To help ensure that employees are able to meet the quality requirements, training standards for crew roles should be developed. The training standards should define a set of training that is required and desirable for various crew member functions. These training standards need to consider the type of work the crews will be performing and the need for specialized skills for crew members.

For example, a bridge painting crew is considered. Every employee working on the crew will require a suite of common health and safety training such as fall protection, confined spaces training and lead health and safety training. These training courses ensure that the employees are working safely but do not address the quality of the work being performed. Training courses related to the proper preparation of steel surfaces for painting, how to monitor weather during painting, and application techniques for paint ensure that the painting will meet the quality standards.

An agency with training standards defined for each role on their crew can compare the current staff training histories against the standards to identify training needs and to assign tasks in a project work plan.

2.5.5 Quality Control in Finished Products

Quality control processes are used to check or test completed work by identifying any defects observed or measured in the finished product. Quality control is a reactive process of sampling and testing to ensure conformance with established quality standards. Quality control is reactive in nature, but can be employed at the completion of intermediate phases of a larger project. For example, the QC activities associated with the placement of new paint on a bridge would certainly include some form of QC of the condition of the metal and quality of the surface preparation before the paint was applied.

It may be necessary to adjust the frequency of verification testing in accordance with the estimated number of QC tests to be performed to space out the testing dates. The rate of verification testing may also vary based on the risks involved. For example, testing structural concrete may be more frequent than testing embankment materials. QC testing may include

visual inspection for workmanship and quality of the finished product, and material testing for adherence to established material specifications.

Quality control efforts require documentation of the results of the inspection, sampling or testing that is done. The agency should specify the minimum level of QC documentation that must be provided in the QMP. Materials or workmanship that do not meet the specified level of quality should be properly documented, including the nature of the non-conformance, location, extent, and disposition (e.g., removed and replaced, reworked, accepted based on engineering judgment, etc.). The documentation is also used to provide feedback for improvement of the quality assurance procedures.

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Chapter 3 - Bridge Anatomy

3.1 Bridge Components and Elements

This chapter provides a general overview of bridge anatomy. For additional information, readers are encouraged to use the FHWA *Bridge Inspector's Reference Manual (BIRM)*. The BIRM is referenced at the end of this chapter.

3.1.1 Introduction

Those involved with bridge maintenance should have a good understanding of bridge anatomy, including the ability to identify and understand the function of the major bridge components and their elements. Most bridges can be divided into three basic components (see Figure 3.1):

- Deck
- Superstructure
- Substructure

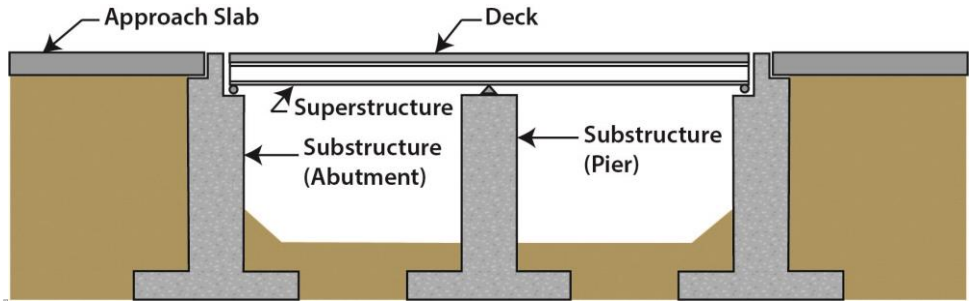


Figure 3.1 Major Bridge Components

3.1.2 Bridge Deck and Related Elements

Elements of the deck are described in the following sections.

3.1.2.1 Structural Deck/Slab

The deck is that portion of a bridge that provides direct support for vehicular and pedestrian traffic. The deck can be constructed from:

- Cast-In-Place (CIP) concrete
- Precast concrete
- Timber
- Steel plate
- Steel grating
- A combination of precast and CIP concrete
- The top flanges of side-by-side precast girders

While normally distributing load to a system of beams or girders, a deck may also be the main supporting element of a bridge. The AASHTO Manual for Bridge Element Inspection uses "slab" to distinguish between slab bridges where the slab transfers loads directly to the substructure and "decks" which transfer loads to the superstructure.

A steel grid deck is shown in Figure 3.2. Steel grid decks are popular for movable bridges since they are lighter in weight making them easier to lift.

Figure 3.2 Steel Grid Deck

Five basic types of timber decks are:

- Timber plank deck (shown in Figure 3.3)
- Nailed laminated deck
- Glued-laminated deck planks
- Stressed-laminated decks
- Structural composite lumber decks

Figure 3.3 Timber Plank Deck

3.1.2.2 Wearing Surface

The “wearing surface” of a deck is the riding surface for the traffic and is placed on top of the structural portion on the deck, as shown in Figure 3.4. There are also wearing courses placed integrally with the structural slab, and then the deck is referred to as a monolithic deck. A wearing surface could be used as a “sacrificial” part of the bridge deck, perhaps milled off and repaved every few years to maintain a smooth surface. Wearing surfaces can provide a barrier to protect the decks from road salts. The wearing surface could just be asphalt, but in that case a membrane might be placed in between the concrete and asphalt interface since asphalt is permeable. In other instances, a more impermeable material might be used.

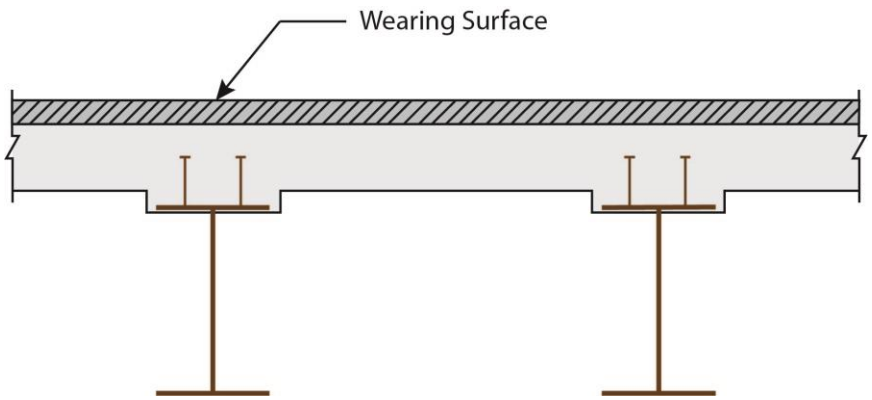


Figure 3.4 Composite Deck and Steel Superstructure Showing Wearing Surface

Materials used as wearing surfaces include:

- Plain concrete
- Modified concretes (polymers, latex, silica fume)
- Asphalt
- Asphalt with waterproofing membrane
- Gravel (tar stabilized or loose)
- Timber treads
- Thin bonded epoxy or methacrylate

When to Call the Engineer

An engineer should be consulted whenever the depth of the wearing surface is increased or the depth of integral concrete wearing surface is decreased due to maintenance operations.

3.1.2.3 Approach Slabs

Most bridge abutments are designed to have very little settlement. Approach embankments usually settle more than abutments. To mitigate this issue approach slabs are used to transition the riding surface from the roadway to the bridge deck. In general, the effectiveness of approach slabs increases with greater length of approach slab. Some approach slabs have secondary approach slabs commonly referred to as “sleeper” slabs.

Approach slabs are reinforced concrete slabs placed on the approach embankment adjacent to and usually resting on the abutment back wall. The function of the approach slab is to carry wheel loads on the approaches directly to the abutment, thereby transitioning any approach roadway misalignment due to approach embankment settlement.

Figure 3.5 Approach Slab to Roadway Approach

3.1.2.4 Joints

Joints accommodate the expansion, contraction, and rotation of the superstructure. Joints provide transitions from approach roadways to bridge decks, or between adjoining segments of bridge decks.

Joints are important to bridge maintenance. Leaking joints are a leading cause of reduced bridge service life. Deteriorating joints are a leading cause of poor wearing surfaces on bridges.

Types of deck joints include:

- Strip seal expansion joints
- Pourable joint seals
- Compression joint seals (see Figure 3.6)
- Assembly joints with seal (modular)
- Open expansion joints
- Assembly joints without seals (finger plate and sliding plate joints) (see Figure 3.7)
- Asphalt plug joints

Figure 3.6 Top View of an Armored Compression Seal

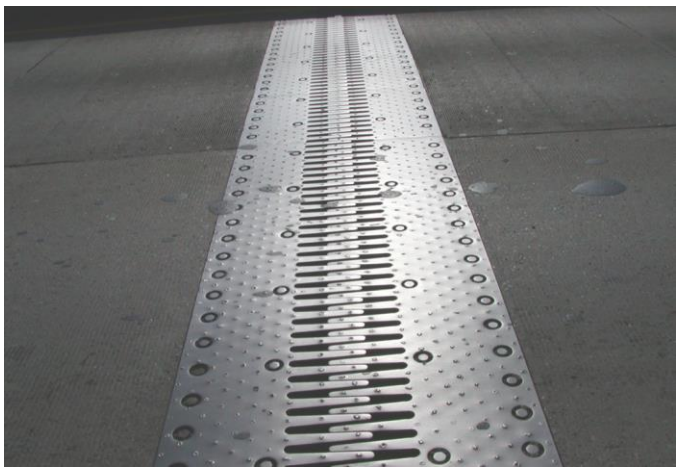


Figure 3.7 Top View of a Finger Plate Joint

Figure 3.8 shows a failed joint where drainage is occurring directly onto the bridge bearings and pier; this situation can cause substantial damage.

Figure 3.8 Leaky Joints Lead to Problems at Bearings and Pier Caps

3.1.2.5 Curbs and Sidewalks

Curbs are provided along sidewalks and safety walks. Curbs can be constructed of reinforced concrete (Figure 3.9), precut granite, timber, or steel plates. Precut granite is popular in areas where there is snowplow usage, since granite can withstand the scraping of the plow blade.

Sidewalks are provided on structures where pedestrian traffic warrants their use. Types of sidewalks are:

- Reinforced concrete
- Steel plate
- Wooden plank

Figure 3.9 Bridge Sidewalk Using Stamped Concrete for Aesthetics and Concrete Curb

3.1.2.6 Railing and Safety Features

Bridge barriers can be broken down into two categories including the bridge railing and the pedestrian railing. The bridge railing is used to guide, contain, and redirect errant vehicles. The pedestrian railing is used to protect pedestrians.

Examples of railings and materials include:

- Timber rail
- Metal angles, tubes, thrie beam bridge railing, and bars
- Combination bridge-pedestrian railing
- New Jersey barrier - a very common concrete barrier

Figure 3.10 End Termination of Metal Beam Guide Rail at Approach Transition

In addition to bridge railings, the transition to the bridge is an important safety feature that should be maintained.

Terms used for the approach guardrail include:

- Approach end treatment
- Approach systems

- Approach transition (see Figure 3.11)

The thrie beam bridge railing system (shown in Figure 3.11) stiffens the typical guardrail and has a larger surface area so vehicles will not get caught underneath.

Figure 3.11 Approach Guardrail Transition

End treatments, such as the example shown in Figure 3.12, are designed to slow and redirect the vehicle from hitting the bridge. They are also designed to absorb energy from the vehicle. Other end treatments include impact attenuators and sand-filled barrels.

Figure 3.12 Example of an Approach Guardrail End Treatment

While railings protect vehicles and pedestrians, another important safety feature on bridges is lighting. Lighting on bridges can consist of travelled way lighting, sign lights, traffic control lights, navigation lights, and aerial obstruction lights. The last two types of lights are special categories which are encountered only on bridges over navigable waterways or on bridges having high towers. Many bridges have no lighting.

3.1.2.7 Drainage

The primary function of a drainage system is to remove water from the bridge deck, from under unsealed deck joints and from behind abutments and wingwalls. Bridge drainage is since ponded water, especially in colder climates, can damage to a bridge or become a safety concern. An effective system of drainage is essential to proper maintenance.

A deck drainage system has the following components:

- Inlets
- Outlet pipes
- Downspout pipes - to transport runoff to storm sewers
- Cleanout plugs - for maintenance
- Drainage troughs
- Support brackets/hardware

Together these drainage components form a deck drainage system. A joint drainage system is a separate gutter or trough used to collect water passing through a finger plate or sliding plate joint.

Substructure drainage allows water in the fill material behind an abutment or wingwall to drain any accumulated water. Substructure drains are weep holes or substructure drain pipes.

3.1.3 Superstructure

The superstructure carries loads from the deck across the span and to the bridge supports. The superstructure is that component of the bridge which supports the deck or riding surface, as well as the loads applied to the deck.

There are many types of superstructure such as:

- Slabs
- Single web beams/girders (I Beams)
- Box beams/girders (Multi-web)
- Trusses
- Arches
- Rigid frames
- Cable-supported bridges
- Movable bridges
- Floating bridges
- Suspension
- Segmental
- Movable (Lift, Bascule, Swing)
- Tee Beam

Schematics of several of these bridge types are shown in Figure 3.13.

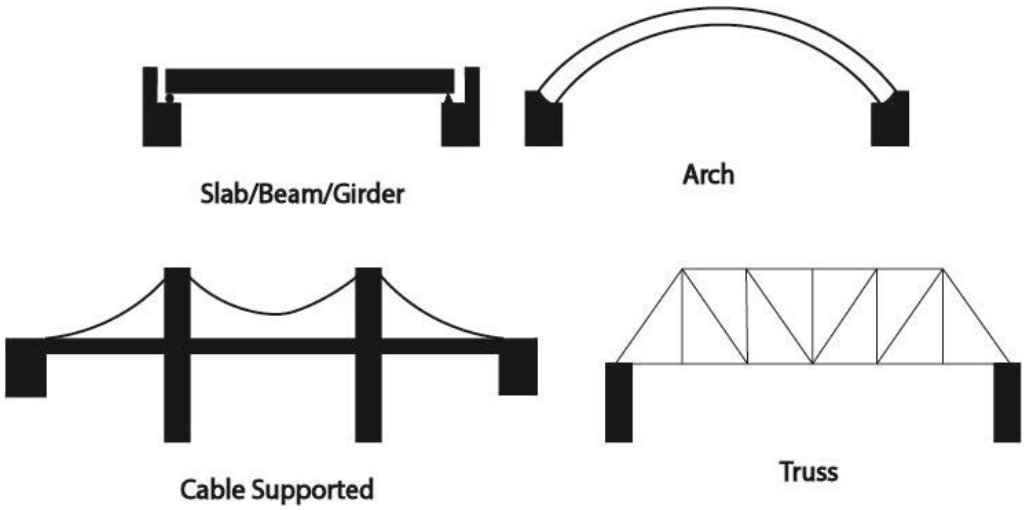


Figure 3.13 Schematics of Several Superstructure Types

3.1.3.1 Beams/Girders

Steel Beams/Girders

The term "steel rolled beams" comes from the "rolling" of the beams in one complete piece. Heated steel blocks are rolled into standard rolled shapes at the steel mill.

Steel built up I-shaped and box-shaped girders offer a great deal of flexibility. They allow the bridge engineer to customize the members for their particular need. Historically, I-girders were fabricated using rivets, plates, and angles. A schematic of a riveted I-girder is shown in Figure 3.14 (left). Riveting was replaced by high strength bolting and welding to produce built up shapes. A built up I shaped girder fabricated from plates welded together is commonly referred to as a Plate Girder. An example of a welded plate girder is shown above in Figure 3.14 (right).

Built up steel shapes can also be used for floor beams, trusses, and substructure members.

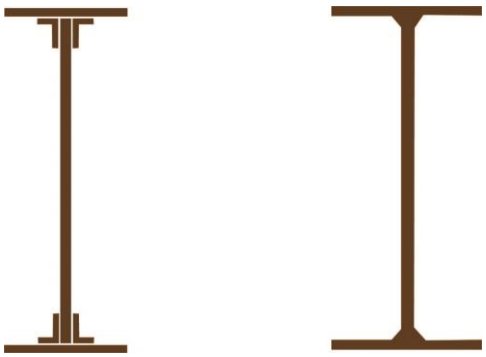


Figure 3.14 Riveted I-girder (left) and Welded Plate Girder (right)

Steel I-shaped and box built up girders are used for intermediate span lengths not requiring a truss or other bridge type capable of very long spans, and yet requiring a member larger than a rolled beam. The elements of these girders are a web plate welded to flange plates. The components of a plate girder are two flange plates, a web plate, and a set of vertical plates attached to the web plate called stiffeners. Stiffeners at concentrated loads are bearing stiffeners. An example is shown in Figure 3.15. Other stiffeners include transverse stiffeners, which improve shear strength of web plates and offer locations for connection of cross frames.

Examples are shown in Figure 3.16. Shear studs are used to provide composite action with concrete bridge decks. Shear studs are also used with rolled beams for this purpose (see Figure 3.17). Composite action will be defined in Chapter 4.

Figure 3.15 Bearing Stiffener

Figure 3.16 Intermediate Stiffeners

Figure 3.17 Shear Studs Being Welded On

Steel box girders can be efficient for curved bridges due to their torsional stiffness; i.e., the ability to resist twisting. Torsion is further discussed in Chapter 4.

Concrete Beams/Girders

Common shapes of reinforced concrete members are:

- Slabs/Decks
- Rectangular beams
- Tee beams
- Channel beams

Schematics of these shapes are shown in Figure 3.18.

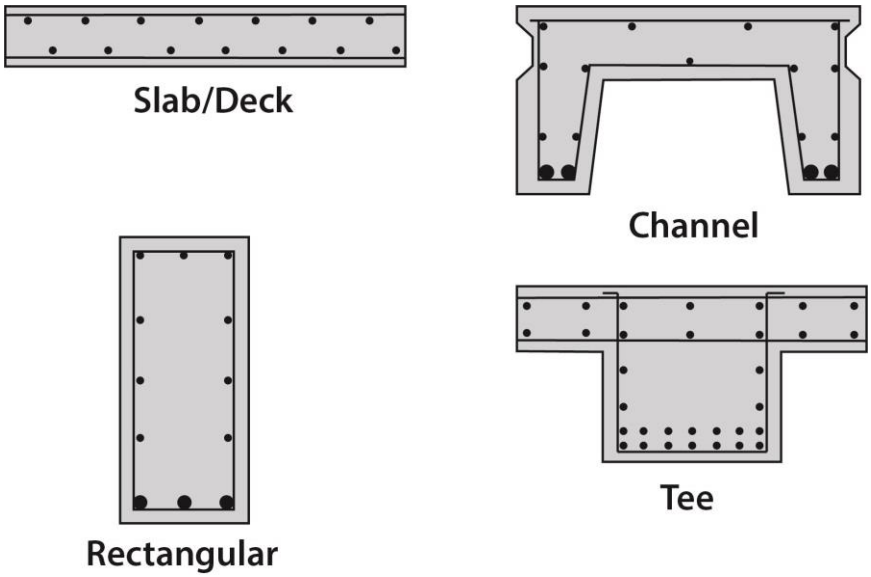


Figure 3.18 Typical Concrete Beam / Girder Shapes

Bridges using these shapes with mild steel reinforcement are often Cast-In-Place (CIP) concrete. Concrete members of this type are used for short and medium span bridges, usually less than 100 feet.

Prestressed concrete girders are produced using high strength concrete and steel tendons. Prestress means that during fabrication, compressive stress is introduced into areas of the beam that normally would be in tension when the beam is subjected to loads. Under loads, those initial compressive stresses are reduced instead of tension stresses being introduced. The prestressing introduces an initial clamping force into the concrete member.

Figure 3.19 shows the relative difference in reinforcing for comparable steel beams. Use of higher strength steel and prestressing requires less tension reinforcement than that required by a conventionally reinforced beam.

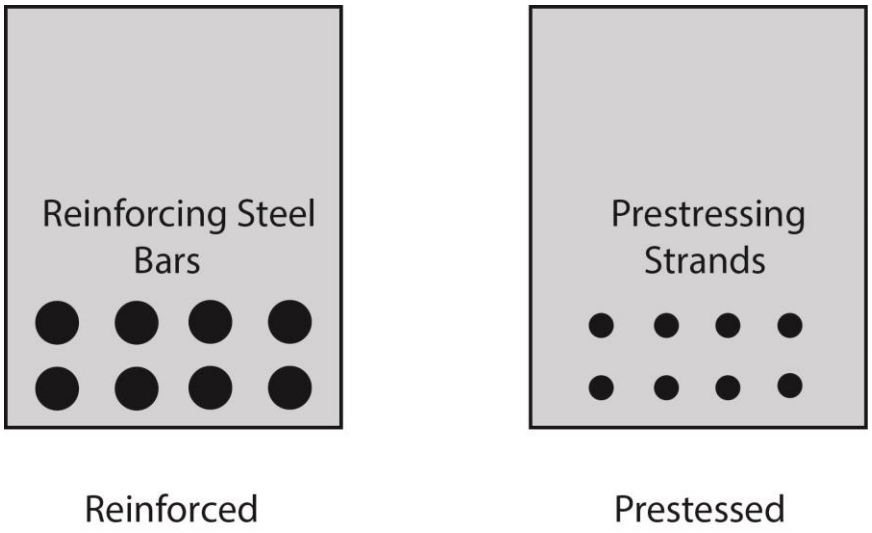


Figure 3.19 Non-Prestressed Reinforced Concrete vs. Precast Prestressed Concrete

See Figure 3.20 for typical prestressed concrete girder sections. The concepts of tension and compression are discussed in Chapter 4 Bridge Mechanics.

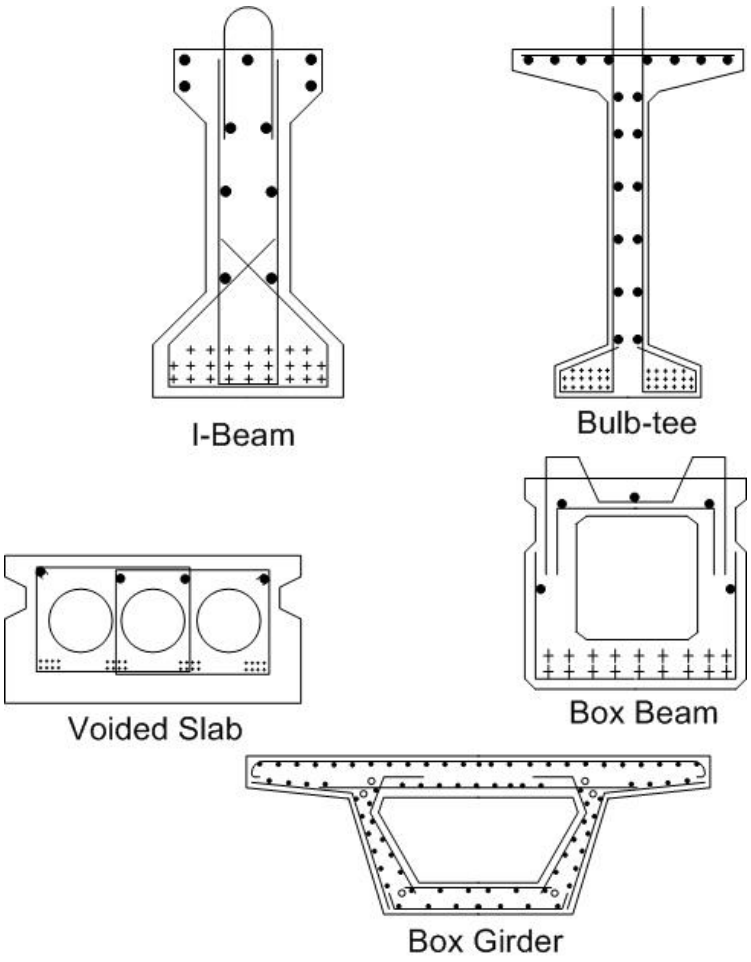


Figure 3.20 Typical Prestressed Concrete Girder Sections

Prestressed concrete is generally more economical than conventionally reinforced concrete due to the benefits of the prestressing, and because prestressing steel is very high strength, fewer pounds of steel are needed. However, prestressed strand repair is difficult.

There are two types of prestressed concrete members: pre-tensioned or pre-stressed and post-tensioned. In a pre-tensioned member, the strands are tensioned before the concrete is placed around them; whereas in a post-tensioned member, the strands are stressed after concrete has cured. For post-tensioning, the concrete is poured first with ducts providing a void for the strands to be passed through later. The strands (rubber band) are then anchored to one end of the girder, tensioned, and anchored at the tensioning end, imposing compression on the girder.

Concrete I-beams, distinguished by their "I" shape, are superstructure members and support the deck. This type of beam is used for spans as long as 150 feet.

Bulb-tee beams are distinguished by their "T" shapes, with a bulb-shaped section (similar to the bottom flange of an I-beam) at the bottom of the vertical leg of the tee. This type of beam has been used for spans as long as 200 feet.

Box beams are distinguished by a square or rectangular shape and internal void, and are typically 18 inches or greater in depth. Box beam bridges may be adjacent or spread, and have a practical span lengths of up to 130 feet.

Box girders, distinguished by their trapezoidal or rectangular box shapes, function as both deck and superstructure. Box girders are used for long span or curved bridges and can be precast and erected in segments or cast in place. Spans lengths can exceed 150 feet with segmental construction.

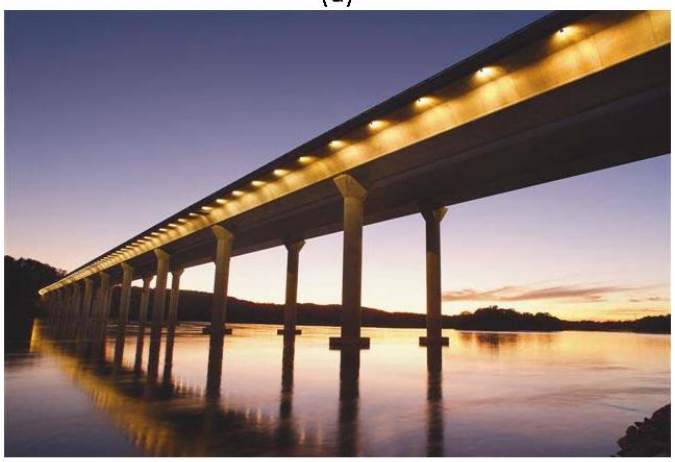
Voided slabs, distinguished by their rectangular shape and their interior voids, are generally precast units supported by the substructure. The interior voids are used to reduce the dead load. Voided slabs can be used for spans up to 60 feet.

A segmental concrete bridge is fabricated piece-by-piece. An example of a segmental bridge is presented in Figure 3.21. The pieces, or segments, are post tensioned together during the construction of the bridge. The superstructure is constructed of precast concrete.

Several characteristics are common to most segmental bridges:

- Used for long span bridges
- Generally comprised of box girder segments
- Each segment is the full width and depth of the bridge
- For very wide decks, many segmental box girders may consist of two-cell boxes or adjacent single boxes
- The lengths of the segments are determined by the available construction methods and equipment

(a)



(b)

Figure 3.21 Segmental Precast Concrete Box Girder Bridge Examples
(a) Precast Segment, (b) Completed Bridge

Timber Beams/Girders

Timber superstructures are still common on secondary and local roads, as well as on forest and logging roads. In 2013, four percent of highway bridge superstructures were timber, and seven percent of highway decks were timber. These include covered bridges found in the northeastern states, as well as timber decks on timber beams, and concrete decks on timber beams. Timber deck bridges are also common, along with timber trusses, arch, and suspension superstructures. Some covered bridges built in the mid-1800s are still in service. Many use a truss framework for the superstructure, although some use arches.

Log beam bridges are constructed from round logs that have been debarked, but are otherwise used in their natural shape. Sawn lumber planks are spiked into the logs to form the roadway surface. In a few cases, soil and rocks are used in place of the planks. A log is sometimes installed at midspan under the log beams to better distribute the live load.

Log beams are generally 20 to 60 feet in length, depending on the length and diameter of available logs. Log bridges are used on logging and low-volume roads, with life cycles of 10 to 20 years.

Sawn lumber beams are fabricated using timbers that usually are 4 to 8 inches wide and 12 to 18 inches in depth. Timber beam bridges employ timber blocking to keep the beams vertical and laterally supported. Common spans are 15 to 25 feet, with a maximum span of 30 feet for general highway use. Longer bridges use intermediate piers to support multiple simple spans of sawn lumber beams. Service life of timber beam bridges can be 40 to 70 years with proper maintenance.

Glulam (Glued-Laminated) timber beams have been available for about 60 years and allow for longer spans up to 140 feet and fewer beams per bridge. Typical spans are 20 to 80 feet.

3.1.3.2 Timber Longitudinal Deck Superstructures

Longitudinal deck superstructures use glulam or nail-laminated sawn lumber or stress laminated sawn lumber with the width of the boards turned vertical to act as both deck and beam in one unit. Typically used with distributor beams at midspan, like the much older log beams, longitudinal deck superstructures are often used for lightweight deck replacements. Similar in application to concrete slab bridges, they have a maximum clear span of 35 feet. These bridges are typically well-suited for crossings where it is desirable to maximize vertical clearance or reduce superstructure dead load.

Nail-laminated deck superstructures use sawn lumber that is either 2 or 4 inch nominal width by 8 to 16 inches deep and is nailed to the adjacent two boards in three alternating patterns repeating on every third board. Rails are often mounted from the underside of the deck or use posts through the edge of the deck by creating “notches” with shorter boards in the nail-lamination.

Figure 3.22 (left) shows the use of a pre-fabricated nail-laminated deck superstructure as a lighter alternative to concrete decks, used here to replace a former open steel grid deck and allow a previously closed single-lane roadway bridge to be re-opened for a multi-use trail bridge. On this bridge, the timbers were not topped by asphalt for a wearing surface due to weight concerns.

An example of a site-constructed nail-laminated bridge is shown in Figure 3.22 (right). The bridge also features the typical waterproof membrane and asphalt pavement to smooth out the uneven edges between the individual timber boards. A note of caution with asphalt overlays is that they tend to crack along the edges of the boards in the laminations, due to expansion and contraction of the wood. These superstructures have been used in other locations to replace roadway bridge superstructures and to extend the life of roadway bridges for vehicular traffic as well.

Figure 3.22 Nail-Laminated Decks. Pre-Fabricated (left) and Site-Constructed (right)

3.1.3.3 Trusses

A truss is a form of structural system that provides high load carrying capacities and can be used to span greater lengths than rolled beams and girders. A schematic of a steel truss is shown in Figure 3.23. Truss members (e.g., chords, verticals, and diagonals) carry axial tension and compression loads. Truss members are selected for their strength in tension or compression, and loads that will introduce bending moment in them should generally not be applied.

Trusses can be constructed from timber or steel, and some smaller truss bridges have been constructed of Fiber Reinforced Polymer (FRP). Usually members in tension are smaller in cross section than compression members. Channels, steel rods or angles are often used for tension members. Examples of compression members include built up girders, latticed channels, or steel box sections.

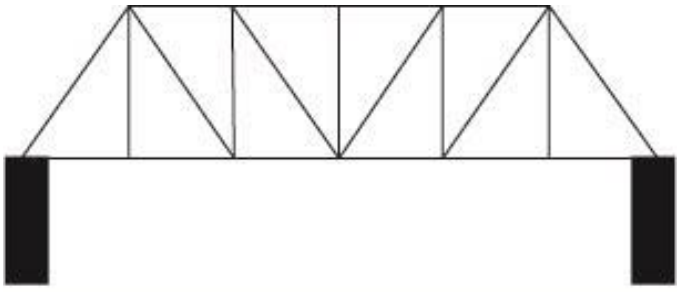


Figure 3.23 Schematic of Steel Truss

Types of structural members in a truss bridge (i.e., primary members) are listed below and are shown in Figure 3.24.

Chords: The upper and lower longitudinal members that extend the full length of a triangular section are called chords. For a simple span, the bottom chord will always be in tension, while

the top chord and end posts will always be in compression. The bottom and top chords are primary structural members. Failure of either chord will make the truss unsafe.

Diagonals: The diagonal members connect successive top and bottom chords and resist either tension or compression depending on the truss configuration.

Verticals: Vertical web members connect the top and bottom chords and resist tension or compression stresses depending on the truss configuration. Most vertical members are primary structural members.

Floor Beams: Floor beams span between the trusses at panel points and carry loads from floor stringers and the deck system to the trusses.

Stringers: Stringers span between floor beams and provide primary support for the deck system. The deck loading is transferred through the stringers to the floor beams and then to the truss.

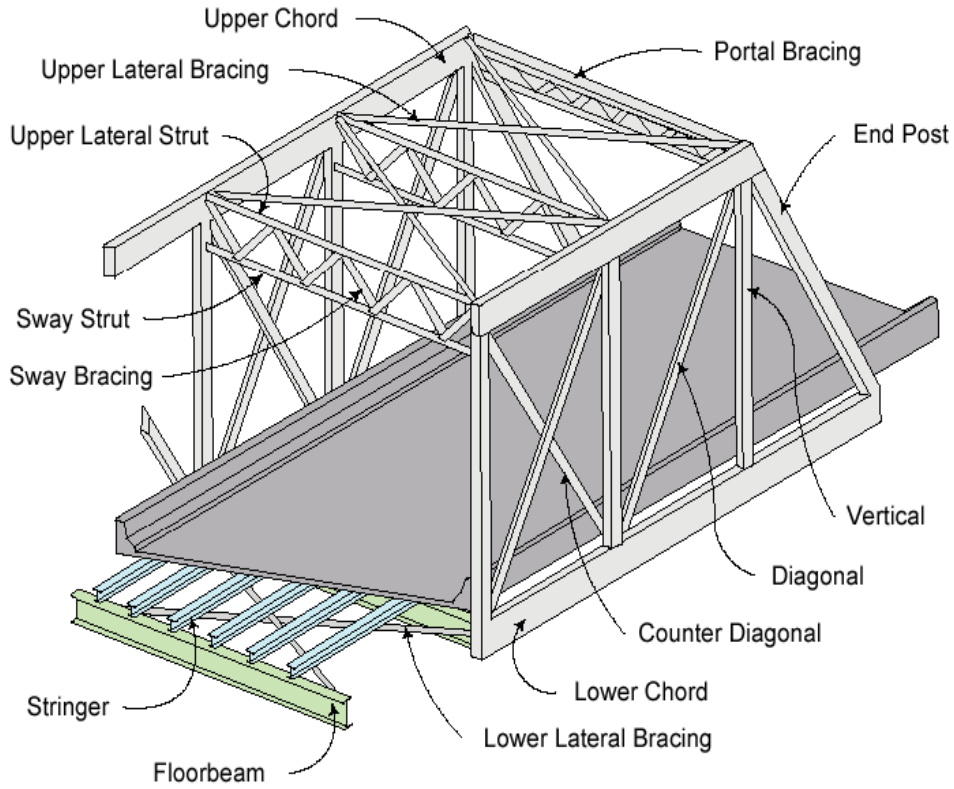


Figure 3.24 Truss Components (Courtesy of ODOT)

There are three types of timber trusses including through trusses, pony trusses, and deck trusses, with many variants in the arrangement of the structural members. Figure 3.25 shows the typical timber truss shapes.

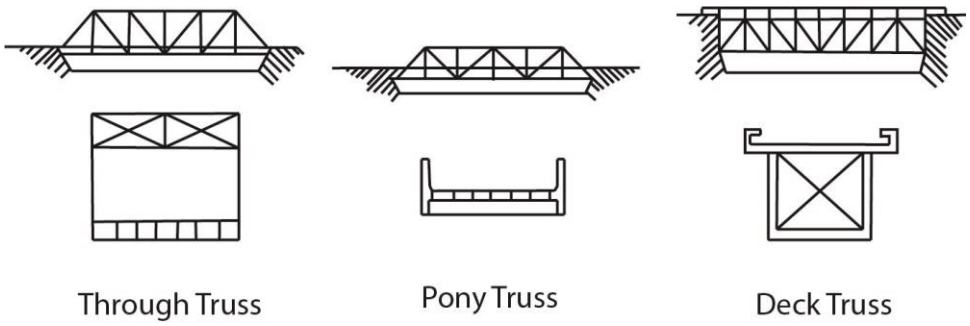


Figure 3.25 Typical Types of Timber Trusses

The bowstring truss (shown in Figure 3.26) was historically constructed with lumber top chords, and later with steel. But with the advent of glulam beams, the top chords have sometimes been replaced by glulam beams which are much easier to curve in the shop into either one-piece or two-pieces per arch.

In a pony truss, the deck is located in-between the top and bottom chord such that there are no top chord cross-braces. Many through trusses look similar to deck trusses that have been vertically flipped. Covered bridges often used Howe, town lattice, queen post, king post, or burr arch trusses for the superstructure.

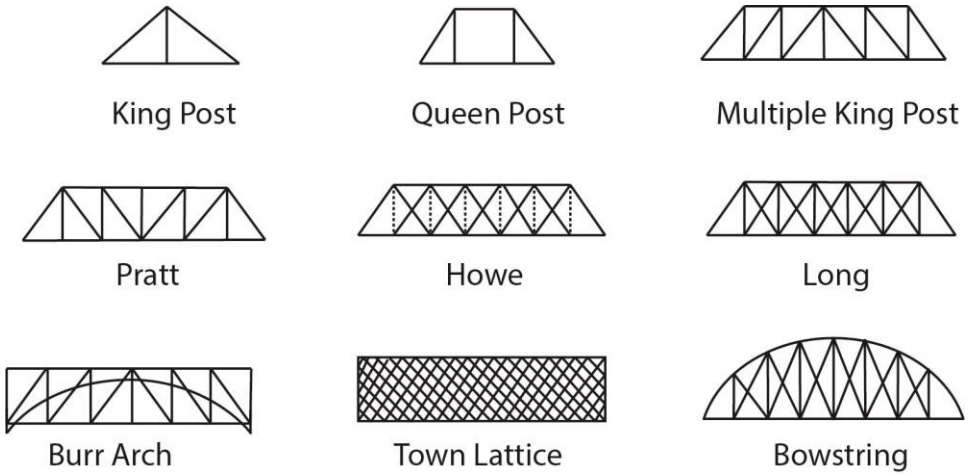


Figure 3.26 Typical Types of Through Trusses

Floor Beam – Stringer System

Stringers span longitudinally between floor beams and transmit loads from the deck to the floor beams. The floor beams span between trusses and transfer loads from the stringers to the trusses, shown in Figure 3.24. Floor beams can also span between longitudinal girders. Floor beams can be used without stringers. In this application the deck spans longitudinally between the floor beams.

Stringers

The term stringers is usually associated with a floor beam system, but sometimes the term is used to describe what has been defined in this manual as beams/girders.

Stiffeners, Bracings, Gusset Plates, and Connection Plates

The individual members of beam and girder structures are tied together with diaphragms and cross frames; trusses are tied together with portals, cross frames, and sway bracing. Diaphragms and cross frames stabilize the beams and trusses and distribute loads among them. A diaphragm is usually a solid web member, either of a rolled or built up shape. An example of a diaphragm is shown in Figure 3.27. A cross frame is a truss, panel, or frame.

Figure 3.27 Diaphragm (K-bracing)

Bearings

Bearings transmit loads from the superstructure to the substructure and, as needed, allow movement due to expansion, contraction, and rotation. Typical of bridge bearings include fixed, rocker, roller, sliding plate, elastomeric, pot, and disc.

A bridge bearing has four basic elements (see Figure 3.28):

1. Sole plate
2. Bearing device
3. Masonry plate
4. Anchor bolts

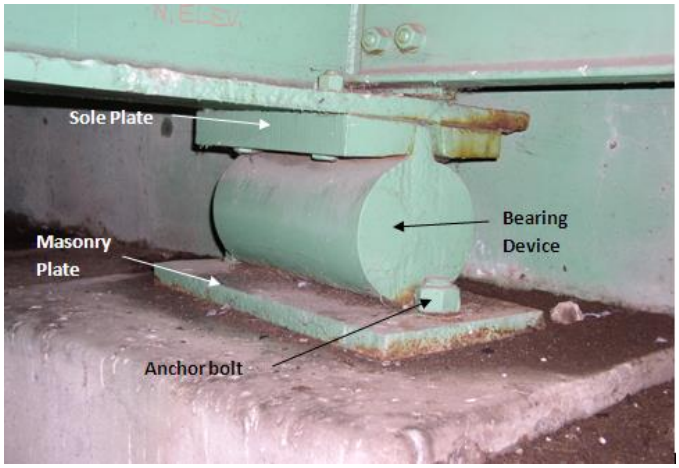


Figure 3.28 Four Basic Elements of a Steel Roller Bearing

Steel roller bearings, shown in Figure 3.28, can need substantial maintenance. In the figure, dirt and debris are observed where the round bearing device comes in contact with the masonry plate. Since the bearing must roll on the masonry plate to allow the bridge to move, dirt and debris may prevent the motion. If that happens the bearing is referred to as “frozen”. If a bearing cannot move, something else must give and that can, for example, create cracks in concrete members. New maintenance-free bearings no longer rely on steel, but on elastomer materials. An example of an elastomeric bearing is shown in Figure 3.29.

Figure 3.29 Elastomeric Bearing

3.1.4 Substructure

3.1.4.1 Abutments

Abutments provide support for the ends of the superstructure and retain the roadway approach embankment. Abutments may be founded on spread footings, piles, or drilled shafts.

Basic types of abutments, shown schematically in Figure 3.30, include:

- Cantilever or full height abutment - extends from the grade line of the roadway or waterway below, to that of the road overhead (see Figure 3.31 and Figure 3.34).
- Stub, semi-stub, or shelf abutment - located within the topmost portion of the end of an embankment or slope. In the case of a stub, less of the abutment stem is visible than in the case of the full height abutment. New construction often uses this type of abutment (see Figure 3.32).
- Spill-through or open abutment - consists of columns and has no solid wall, but rather is open to the embankment material. The approach embankment material is usually made of rock.
- Integral abutment – superstructure and substructure are integral and act as one unit without an expansion joint or bearings. Relative movement of the abutment with respect to the backfill allows the structure to adjust to thermal expansions and contractions. Pavement relief joints at the ends of approach slabs are provided to accommodate the thermal movement between bridge deck and the approach roadway pavement (see Figure 3.33).
- Semi-integral abutment - similar to integral abutments, however, the superstructure and the top of the abutment act as one unit, but the bottom portion act independently of the superstructure. This is achieved by a joint between the top and bottom portions

of the abutment which will allow for un-restrained rotation and thermal movement (see Figure 3.31).

- Mechanically Stabilized Earth (MSE) abutment (see Figure 3.34 and Figure 3.35).

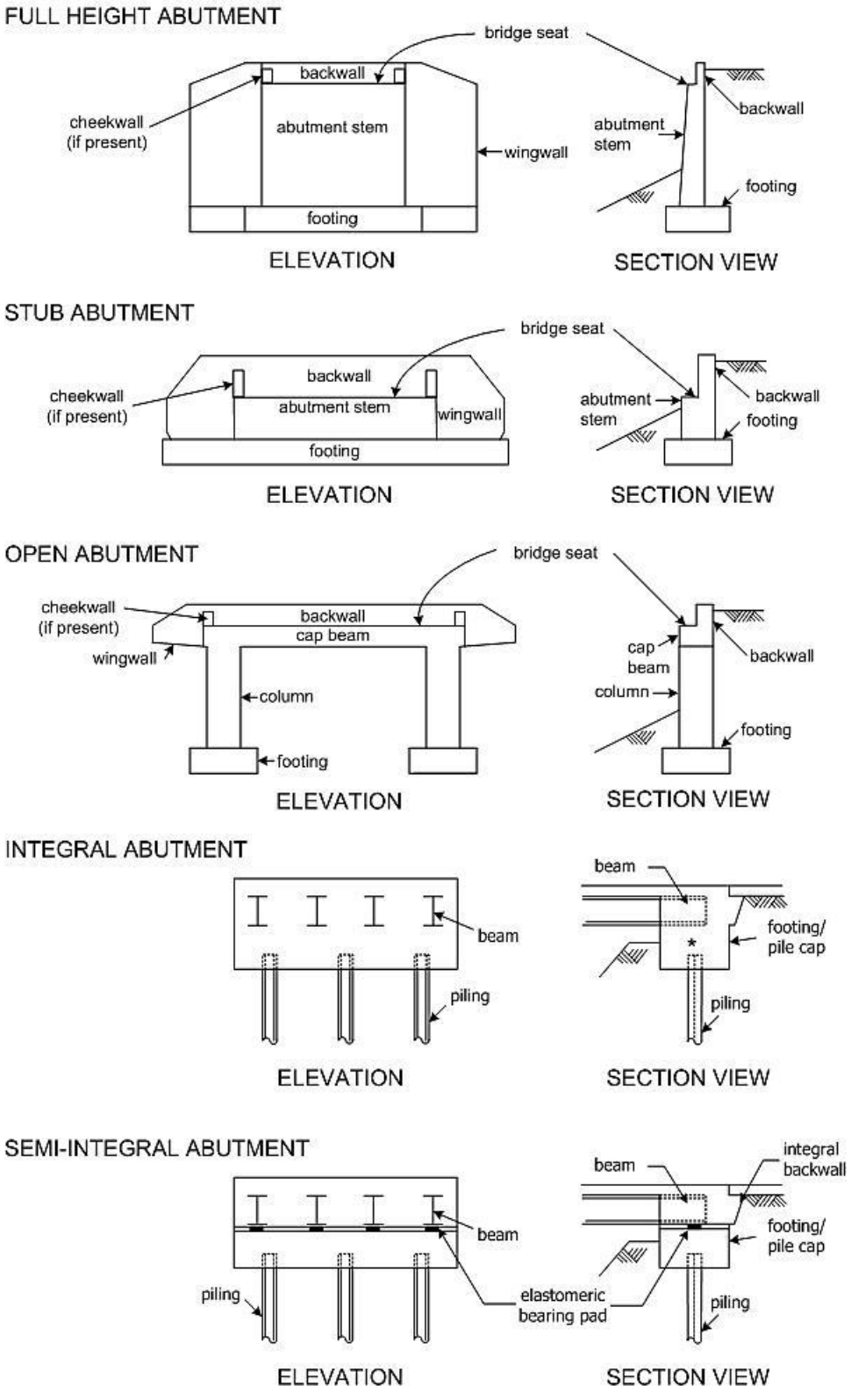


Figure 3.30 Schematics of Several Abutment Types

Figure 3.31 Abutment Supporting Superstructure End

Figure 3.32 Example of a Shorter Abutment

Figure 3.33 Example of an Integral Abutment

Figure 3.34 Abutment Utilizing Mechanically Stabilized Embankments (MSE)

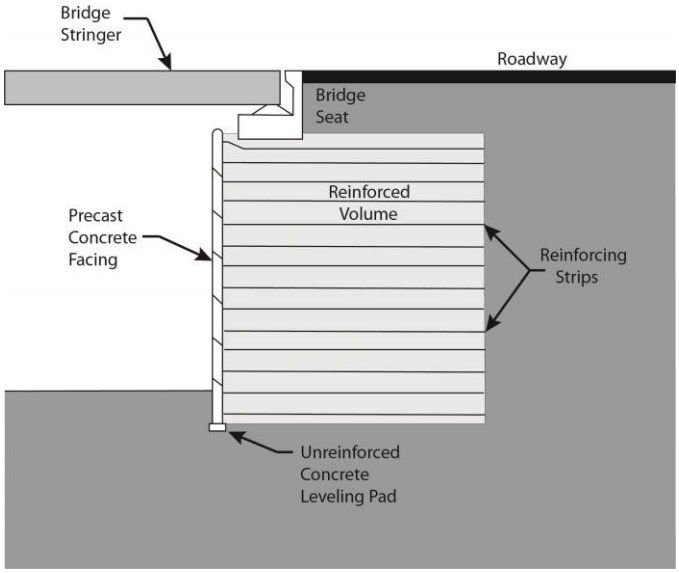


Figure 3.35 Concept of MSE Abutment

3.1.4.2 Wingwalls

Wingwalls are the retaining wall extension of an abutment. Their purpose is to retain side slope material of an approach roadway embankment. An example of a wingwall is shown in Figure 3.36.

Figure 3.36 Example of a Wingwall

3.1.4.3 Piers

Bridge piers, shown in Figure 3.37, transmit the load of the superstructure to the foundation material and provide an intermediate support between abutments.

Figure 3.37 Piers Support the Superstructure

A pier has only one footing at each substructure unit (the footing may serve as a pile cap). A bent has several footings or no footing, as is the case with a pile bent. There are several basic types of piers:

- Solid shaft pier (see Figure 3.38 and Figure 3.39)
- Column pier (see Figure 3.40)
- Column pier with web wall
- Cantilever or hammerhead pier (see Figure 3.41)
- Column bent
- Pile bent (see Figure 3.42)

Figure 3.38 Solid Shaft Pier

Figure 3.39 Solid Shaft Pier

Figure 3.40 Column Pier

Figure 3.41 Hammerhead Pier

3.1.4.4 Caps

Caps are the topmost portion of a pier or a pile bent. Their purpose is to (1) distribute the loads to the columns or piles and (2) to hold columns or piles in their proper relative positions. Caps are usually horizontal members and can be made of concrete, steel or timber. An example of a concrete cap is shown in Figure 3.42. See sections on “Piers” and “Piles” in this chapter for more information and photos.

Figure 3.42 Concrete Cap as Part of Pile Bent Structure

3.1.4.5 Piles

Piles are shaft-like members that carry load to the underlying rock or soil strata. A lead-on pile driver holds the pile-in-place and also guides the piles into the ground at the proper angle, as shown in Figure 3.43.

Figure 3.43 Lead-on Pile Driver

When to Call the Engineer

An engineer should be consulted when piles normally driven into the ground become exposed by scour or erosion, since the piles can become unstable.

Most piles are buried, so not visible. However, a pile bent is a row of driven or placed piles extending above the ground surface supporting a pile cap, as shown in Figure 3.44.

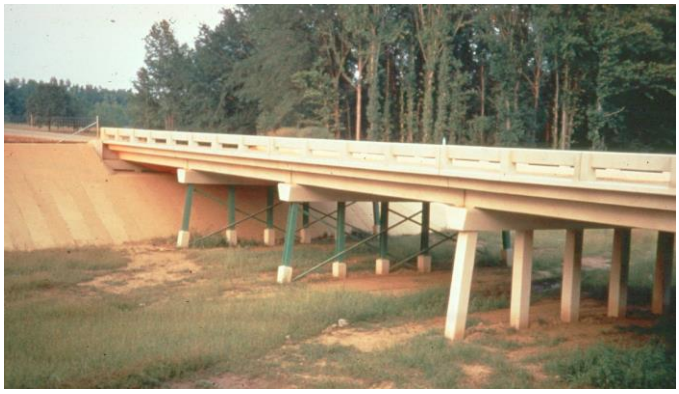


Figure 3.44 Example of Pile Bents

Types of piles include:

- Steel H-piles
- Steel pipe piles (sometimes filled with concrete)
- Timber piles
- Prestressed concrete piles
- Reinforced concrete cast in place into pre-drilled holes (also called drilled shafts)

3.1.4.6 Columns

A column is a general term applying to a vertical member having, in general, a considerable length in comparison with its transverse dimensions. A column transfers load from the superstructure into the ground. The term column and pier are sometimes used together to describe the element, see section on “Piers” for more information.

3.1.4.7 Footings

Footings are the enlarged, lower portion of a substructure that distribute the structure load either to the earth or to supporting piles. A common footing is the concrete slab.

3.1.5 Structures Located in the Channel

Rivers are dynamic systems. During floods, significant changes can occur in a short period of time. There are several ways in which channels can change and thereby jeopardize the stability and safety of bridges. For this reason, structures are built in the channel to control this potentially dynamic system. On the other hand, structures are also placed in the channel to protect the bridge, usually the substructure. Both of these structure types are described in greater detail below.

3.1.5.1 Fender Systems

Fender systems are structures that act as a buffer to protect bridge piers exposed to collision by floating debris and water-borne traffic. Fenders are called ice guards in regions with ice floes. A fender pier is a pier-like structure which performs the same service as a fender but is generally more substantially built. An example is shown in Figure 3.45.

Figure 3.45 A Bridge Fender System

3.1.5.2 Dolphins

Dolphins are a group of piles driven close together or a caisson placed to protect portions of a bridge exposed to possible damage by collision with river or marine traffic. Examples are shown in Figure 3.46.

Figure 3.46 Examples of Dolphins. Timber Dolphin (left) and Steel Pipe Pile Dolphin with Concrete Cap (right).

3.1.5.3 Slope Protection

Slope protection is a surfacing of stone, concrete or other material deposited on a sloped surface to prevent its disintegration by rain, wind or other erosive action; also known as slope pavement. Slope protection can consist of the placement of geotextiles, wire mesh, riprap, paving, revetment, plantings or other materials on channel embankment. The purpose is to protect the slope from erosion, slipping or caving or to withstand external hydraulic pressure.

Figure 3.47 is an example of slope protection using a gabion basket. A gabion basket is a wire mesh filled with rocks. Gabions can be stacked. A common type of slope protection for bridges is installation of riprap, or large stones, placed along the slope. Figure 3.48 is a photo showing riprap protecting a bridge abutment.

Figure 3.47 An Example of a Gabion Basket Serving as Slope Protection

Figure 3.48 Riprap Used for Slope Protection

In addition to gabions and riprap, there are other ways to protect a channel. Some include poured concrete, articulated blocks, and vegetation. These protection methods are covered in greater detail in Chapter 15.

3.1.5.4 Spurs and Dikes

Spurs and dikes are used to prevent scour or redirect water away from bridges. A spur is a projecting jetty-like construction placed adjacent to an abutment or embankment. A dike is an earthen embankment constructed to retain or redirect water. When used at a bridge, the dike prevents stream erosion and localized scour. As a result, the stream current is directed such that debris does not accumulate.

3.2 Bridge Types and Design Details

3.2.1 Multi-Girder System Bridge A multi-girder bridge is a popular type of bridge. The girders, as described in Section 3.1, can be concrete, steel or timber. Examples of timber and concrete multi-girder bridges are presented in Figure 3.49 and Figure 3.50. An attractive feature of the multi-girder system is that there is redundancy; if one girder cannot hold its load there are others that can share the load. The disadvantage is that span lengths are limited, and they have more bearings to maintain than a two girder or truss bridge.

Figure 3.49 Timber Multi-Girder Bridge

Figure 3.50 Prestressed Concrete Multi-Girder Bridge

3.2.2 Girder and Floorbeam Bridge

A girder and floorbeam bridge is rarely used in new construction since there is no redundancy. If a girder failure is imagined in the example 2-girder system bridge shown in Figure 3.51, the entire bridge could collapse. This is in contrast to the multi-girder bridge that has redundancy; failure of one girder does not cause immediate failure of the bridge. The advantage of this type of bridge is that there are fewer bearings to maintain compared with a multi-girder bridge.

Figure 3.51 2-Girder Steel Bridge

3.2.3 Thru-girder (Concrete and Steel)

A thru-girder bridge consists of two girders on the outer edges of the roadway. The bridge deck is supported by floor beams that span between these two girders. These types of bridges are most commonly used for railroads since very heavy loadings can be supported. They are also used in locations where distance between the top of the roadway and bottom of the structure is restricted. However, similar to the 2-girder bridge, this system lacks redundancy. Thru girders are also exposed to possible vehicle impacts, as well as exposure to precipitation and de-icing chemicals in colder climates. Figure shows a thru-girder bridge.

Figure 3.52 Thru-girder Bridge

3.2.4 Slabs

In bridges, loads from the slab are transmitted vertically to the substructure without the use of girders or beams. As a result, slab bridges are used for short span bridges, typically less than 30 feet. The entire superstructure of a concrete slab bridge can be constructed in one pour. Similarly, the timber planks become the entire superstructure for a timber slab bridge. The advantage of slab bridges is that they are relatively simple to construct, but they are limited in the distance they can economically span.

3.2.5 FRP Decks and Superstructures

Fiber Reinforced Polymer (FRP) materials have been used in bridge construction for more than 25 years, mostly for demonstration and research projects. However, they are becoming more widely accepted as a substitute to concrete, timber, and steel. FRP materials are very strong, much stronger than concrete on a strength to weight basis, and do not rust like steel or decay like timber. FRP materials have been successfully used in deck replacements where a lighter weight deck was needed, in pedestrian bridges and bridges in remote locations where transporting large girders could be a problem. FRP is used for various repairs and for strengthening of bridges.

3.2.6 Rigid Frames

Rigid superstructures are characterized by rigid (moment) connections between the horizontal girder and the legs. Legs are the rigid vertical supports with moment connections to the other portions of the superstructure. This connection allows the transfer of both axial forces and moments into vertical or sloping elements, which may be classified as superstructure or substructure elements depending on the exact configuration. Similar to beam/girder or slab configurations, rigid frame systems may be multiple parallel frames or may contain transverse floor beams and longitudinal stringers to support the deck. The advantages of rigid frame bridges are reduced structure depth at midspan and reduced space requirements for approaches/abutments. The disadvantages include that they are less tolerant of settlement than other types of bridges. An example of a bridge with a rigid frame is shown in Figure 3.53.

Figure 3.53 Example of a Rigid Frame Bridge

3.2.7 Trusses

There are a great variety of truss systems in existence, since engineers from the beginning of modern bridge building have sought the perfect weight-to-load carrying proportions. In general, truss nomenclature is usually related to the manner in which the truss provides support relative to the deck. For a deck truss bridge (see Figure 3.54) the truss supports the bridge deck from underneath. For a through truss bridge, the truss support is provided from above the bridge deck (see Figure 3.55). Trusses have the advantage of spans up to 1300 feet, but they also have the disadvantage of non-redundancy. Many trusses also have connections that are prone to trapping moisture and debris.

Figure 3.54 Deck Truss Bridge

Figure 3.55 Through Truss Bridge

3.2.8 Arches

Similar to truss bridges, arch bridges have been around a long time, and the nomenclature is similar. Arch bridges are still constructed today for spans up to 1800 feet. The arch handles large compressive forces and the vertical components can be either in tension or compression depending on the arch type. A deck arch bridge has vertical spandrels, as shown in Figure 3.56, that carry loads from the deck to the arch. A through arch bridge uses tension members, e.g., steel cables, to support the roadway deck. Arches have the advantage of being able to span longer distances, but have the disadvantage of being redundant.

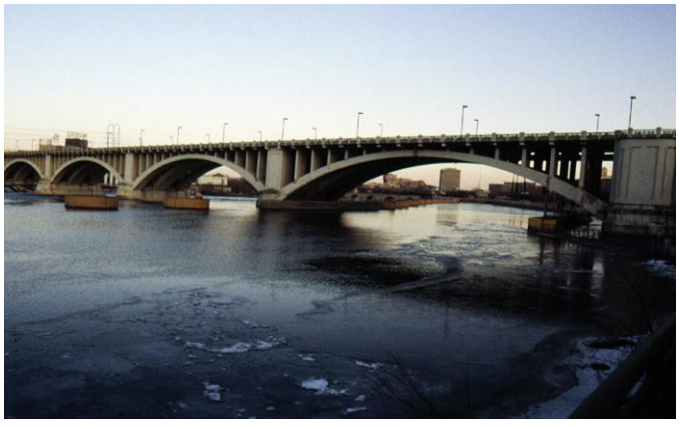


Figure 3.56 Deck Arch Bridge

Glulam suspension arches have allowed timber to enter into the suspension market as an alternative to steel arches. Arches made of glulam can span crossings up to 200 feet, and can be parabolic or segmental circular in shape. There are two main types: two-hinge arches hinged only at the abutments and three-hinge arches, where the third hinge is located at the top of the arch at midspan. Roadways can be connected by glulam posts or steel cables to the arch.

An example of a three-hinge glulam suspension arch bridge is shown in Figure 3.57. Notice the third hinge is a steel pin at mid-span on the arch. Glulam floorbeams support the paved two-lane road above, with the sidewalk supported by cantilevering the glulam floorbeams on the arch.

Figure 3.57 Glulam Suspension Arch Bridge

Glulam deck arches had their start in Oregon in the 1940's but have since grown to be used throughout the country. Glulam deck arches offer an alternative to steel deck arches, especially in regions where timber is plentiful and glulam production is nearby. Often these appear as trestle-like timber frames above the arch as the bottom chord, providing tall clear spans over highways and deep waterways. Keystone Wye Interchange Bridge in South Dakota is a well-known example of a glulam deck arch bridge, as shown in Figure 3.58. In general, glulam deck arch bridges have been used to span deep valleys. Similar to glulam suspension arches, glulam deck arch bridges can be two-hinged or three-hinged.

Figure 3.58 Glulam Deck Arch Bridge

3.2.9 Culverts and Pipes (Concrete and Metal)

3.2.9.1 Introduction

Culverts are small bridges that are constructed entirely below and independent of the roadway surface. Culverts do not have a deck, superstructure, or substructure. A culvert is usually a hydraulic structure, and its main purpose is to transport water flow efficiently, but can also be used for pedestrian, vehicle, or animal underpasses. An example of a culvert is shown in Figure 3.59.

There are several common materials used in the construction of culverts:

- Concrete
- Masonry
- Steel
- Aluminum
- Timber
- Plastic

A culvert takes advantage of submergence to increase water carrying capacity.

Figure 3.59 Culvert Structure

3.2.9.2 Headwalls

Headwalls are used to retain the fill, resist scour, and improve the hydraulic capacity of the culvert. Headwalls are usually reinforced concrete, as shown in Figure 3.60, but can be constructed of timber or masonry. Metal headwalls are often found on metal box culverts.

Figure 3.60 Reinforced Concrete Culvert Headwall and Wingwalls

The common types of headwalls are:

- Projecting: The barrel simply extends beyond the embankment (see Figure 3.61). No additional support is used.
- Mitered: The end of the culvert is cut to match the slope of the embankment. This is used when the embankment has slope paving (see Figure 3.62).
- Skewed: Culverts, which are not perpendicular to the roadway, may have their ends cut parallel to the roadway (see Figure 3.63).
- Pipe end section: A section of pipe is added to the ends of the culvert barrel. These are used on smaller culverts.

Figure 3.61 Culvert End Projection

Figure 3.62 Culvert Mitered End

Figure 3.63 Culvert Skewed End

3.2.9.3 Wingwalls

Wingwalls and headwalls support the embankment around the openings of the culvert, as shown in Figure 3.64. This is the same function that an abutment wingwall provides for a bridge.

Figure 3.64 Cast-In-Place Concrete Headwall and Wingwall

Culverts can be rigid, flexible, or something in between. All culvert types carry some loads by soil arching when the fill depth is sufficient. When there is no fill or the fill is shallow, rigid culverts independently support all loads. An example of a rigid culvert is shown in Figure 3.65.

Figure 3.65 Rigid Culvert

Flexible culverts require lateral earth pressure to help maintain their shape and soil arching to carry loads. An example of flexible culverts is shown in Figure 3.66. The loads are distributed through the flexible culvert and backfill. Backfill is critical to the performance of a flexible culvert.

Figure 3.66 Flexible Culverts

3.2.9.3 Box Culvert

One type of rigid culvert is the concrete box culvert as shown in Figure 3.67. Box culverts are used in a variety of circumstances for both small and large channel openings and are easily adaptable to a wide range of site conditions, including sites that require low profile structures. In situations where the required size of the opening is very large, a multi-cell box culvert can be used, as shown in Figure 3.68. A box culvert may have multiple barrels, but it is still a single structure. The internal walls support the top slab.

Figure 3.67 Concrete Box Culvert

Figure 3.68 Multicell Concrete Box Culvert

Concrete box culverts can be cast-in-place or precast. Precast concrete box culverts are generally preferred for speed of onsite construction. For situations with complex site geometries or other special applications, cast-in-place concrete box culverts may be the preferred choice.

3.2.9.4 Barrel/Pipe

Concrete Pipe Culverts are precast. Precast concrete pipe culverts are manufactured in three standard shapes:

- Circular
- Horizontal elliptical
- Vertical elliptical

Circular pipe culverts are very common. In situations where the required size of the opening is very large, two or more concrete pipe culverts may be used, as shown in Figure 3.69.

Figure 3.69 Twin Concrete Pipe Culvert

Elliptical shapes are used in situations where horizontal or vertical clearance is limited. The oblong shape allows the pipe to fit where a circular pipe would not, but still allows for the necessary size opening. Elliptical shaped pipe culverts may also be used when a wider section is desirable for low flow levels.

3.2.10 Cable Supported Bridges

Types of cable supported bridges include suspension, cable stayed, and tied arch.

A suspension bridge uses cables to suspend the roadway deck and, in simple terms, acts like an old fashioned clothes line, as shown in Figure 3.70. A drawback of a suspension bridge is the need to anchor the clothes line, which is costly. Another type of suspension bridge uses a "self-anchored" approach where the cables are anchored to the ends of the deck, making the deck not only a riding surface but also a larger compression element.

A suspension bridge has a deck which is supported by vertical suspender cables that are, in turn, supported by main suspension cables. The suspension cables are supported by saddles atop the towers and are anchored at their ends.

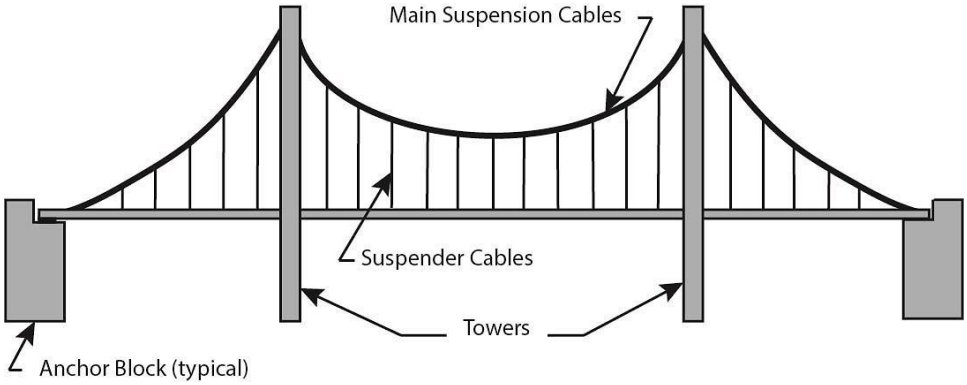


Figure 3.70 Main Elements of a Cable- Supported Suspension Bridge

An example of a cable-supported suspension bridge is shown in Figure 3.71.

Figure 3.71 Cable-Supported Bridge: Suspension Cables and Suspenders

In cable stay bridges, the superstructure is supported by cables or stays that are anchored to towers or pylons located at the main piers. The two major types of cable stay bridges are fan and harp (i.e., parallel), as shown by the schematic Figure 3.72.

Figure 3.72 Types of Cable Stay Bridges. Fan (left). Harp (right).

An example of a cable stay bridge is shown in Figure 3.73.

Figure 3.73 Cable-Supported Bridge: Cable Stayed

The tied arch bridge is one where the reactive horizontal forces acting on the arch ribs are supplied by a tension tie at deck level of a through or half-through arch (see Figure 3.74).

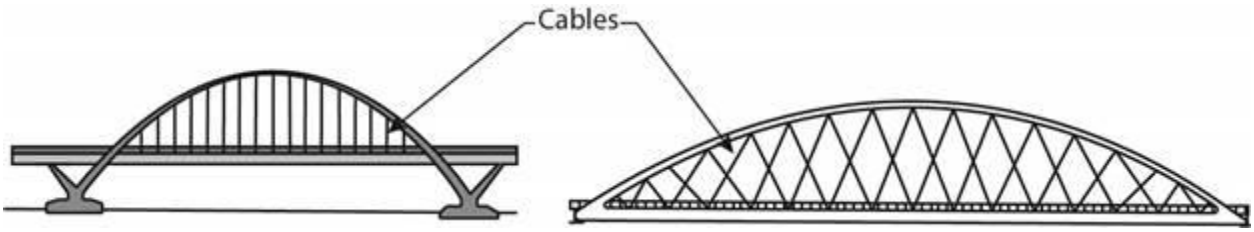


Figure 3.74 Steel Arch with Vertical (left) and Diagonal (right) Hangers

Steel cables are tension members and are used in suspension, some tied-arch, and cable-stayed bridges. Examples of different types of cable cross-sections are shown in Figure 3.75. They are used as the main cables and hangers of this bridge type. The advantage of cable supported bridges is their ability to be used for the longest spans, up to 6500' for suspension bridges and 3600' for cable stayed. The disadvantage is a relatively light and flexible deck which can be subject to vibration in high winds if not stiffened or shaped to be aerodynamically stable.

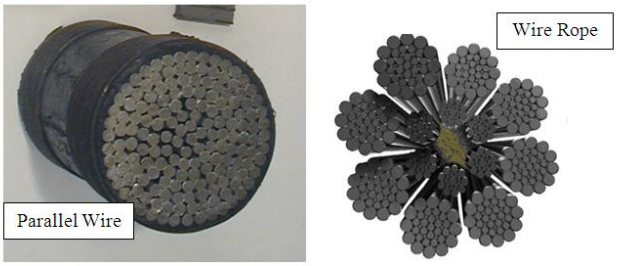


Figure 3.75 Examples of Cable Cross-Sections

3.2.12 Ancillary Structures

Ancillary structures are secondary structures on the highway system, meaning secondary to bridges. Bridges are the main focus of highway structure inspection, maintenance and repair, but recent failures of secondary structures have gotten the attention of the FHWA. Large overhead sign structures have failed and collapsed to the roadway injuring and killing motorists, as have high mast lights, luminaires and retaining walls. Ancillary structures include:

- Structures that support signs
- High mast lighting
- Retaining walls
- Sound walls
- Traffic Signals
- Utilities

Chapter 19 of this manual covers ancillary structures such as bridge mounted signs and utilities.

Figure 3.76 Failed Ancillary Sign Structure

3.3 Chapter 3 Reference List

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Chapter 4 - Bridge Mechanics

4.1 Introduction

The information presented in this chapter is an overview of bridge mechanics. For additional information, readers are encouraged to use the FHWA Bridge Inspector's Reference Manual (BIRM). The BIRM is referenced at the end of this chapter.

Bridge mechanics means the way in which forces act on a structure, and how the structure responds under those loads. The schematic in Figure 4.1 illustrates an archetypal bridge. The symbols used for the bearings indicate that it is a single-span, simply supported bridge, with one fixed-end (the triangle) and the other end free to roll or slide (the circle).

There are many types of structures, each of which can have a numbers of spans and configurations, but this simple figure will be suitable for illustrative purposes. The sketch illustrates the concept of *load path*, the direction that forces take when transmitting the weight of a truck to the ground beneath it. The weight of a single truck is applied to the top surface, the deck, and then passed on to superstructure elements that support the deck. The primary longitudinal structure members carry the loads to the ends of the bridge where it is carried through bearing to rest on the abutments. Abutments distribute the weight of the bridge and vehicles to the ground under the abutment footing. A spread footing spreads the load directly to the earth, but if a bridge is built on end-bearing piles, the footing transfers the loads deeper into the ground where it is supported by rock or other dense material.

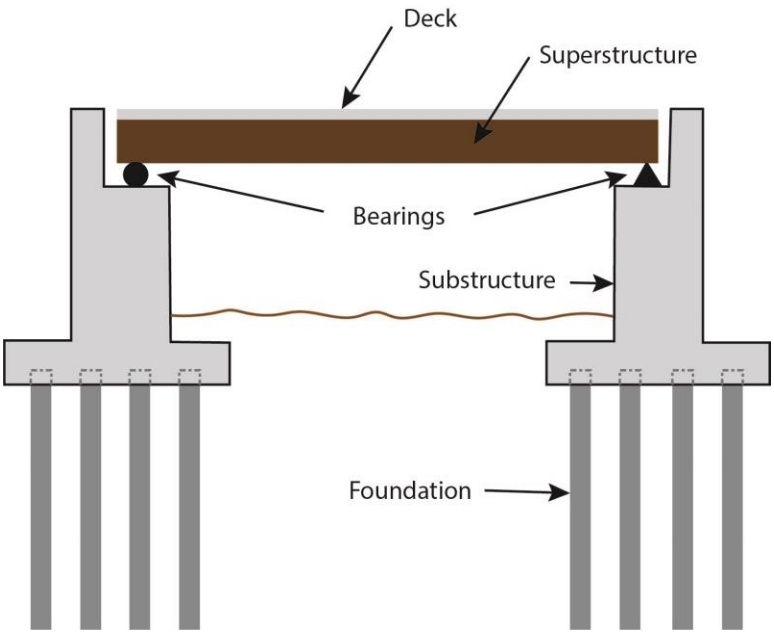

Figure 4.1 Schematic of a Single-Span Bridge Supported by Piles

The rest of this chapter uses specific types of bridges to illustrate certain broad concepts, but it should be recognized that there are many types of structures, and how a bridge carries loads depends on the specific structure.

Culverts are given a dedicated section below because, although they perform the same function as a bridge, culverts are structurally different from bridges. Culverts can be of various shapes and materials, but in most cases they rely on surrounding soil to assist in distributing loads from above to the ground underneath.

4.2 Basic Structural Mechanics

Bridges are subject to forces from many sources such as vehicular traffic, wind, seismic, ice flow, and temperature. Vehicular traffic is referred to as live load and is considered static. The bridge must be able to hold up its own weight, which is referred to as dead load. Dead loads also include permanent attachments such as signs and luminaires and utilities. Additional types of load include water, wind, and extreme-event (earthquake shift) loads and frictional forces. Each of these loads translates into a type of member force.

In bridge design, there are five member forces that are considered:

1. Axial tension
2. Axial compression
3. Shear
4. Flexure
5. Torsion

As a bridge maintenance worker, it is important to understand the types of forces and how forces affect a bridge member.

4.2.1 Axial Tension

Axial tension is a force that acts along the longitudinal axis of the member and tries to pull it apart. Figure 4.2 illustrates axial tension. The word “axial” is used because the force is straight down the middle of the member, so there is no bending or twisting.

Figure 4.2 Axial Tension

Imagine a bridge member in a tug of war; this is a simplistic but effective image. An example where axial tension is the only force is in bridge cables, such as a suspension bridge. Figure 4.3 illustrates axial tension in a suspension bridge. Tension forces are also present in the cables of a cable-stay bridge, as shown in the schematic Figure 4.4.

Figure 4.3 Axial Tension in a Suspension Bridge

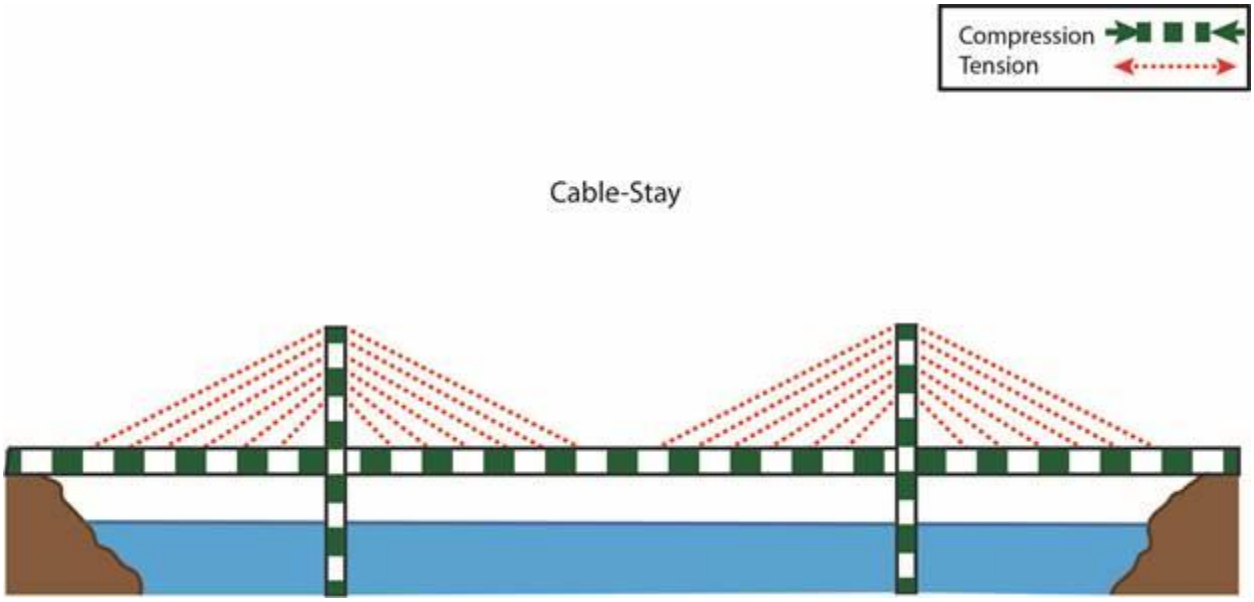

Figure 4.4 Axial Tension in a Cable-Stay Bridge

Most bridge members are subject to multiple forces, so axial tension is rarely the only force acting. Tension members are fabricated from materials that have high strength in tension (i.e., steel) which is not only strong in tension, but also compression and bending. This is why steel is used for compression and bending members, as well as tension members. Concrete is strong in compression, but weak in tension, so steel reinforcement (i.e., rebar) is embedded in the concrete to handle tension.

4.2.2 Axial Compression

Axial compression is a force that also acts along the longitudinal axis of a member, but pushes the member together. This is the exact opposite of axial tension. Figure 4.5 illustrates axial compression. Axial compression is a crushing force, such as an aluminum can being crushed. An

example of bridge members subject to axial compression are the concrete columns shown in Figure 4.6. Concrete is ten times as strong in compression compared to tension, so many bridge components in compression are made of concrete. The concrete column shown in Figure 4.6 has other forces acting on it besides axial compression. For example, the bridge deck expands and contracts during temperature fluctuations which may cause bending in the column. An example of axial compression in an arch bridge is shown in the schematic in Figure 4.7.

Figure 4.5 Axial Compression

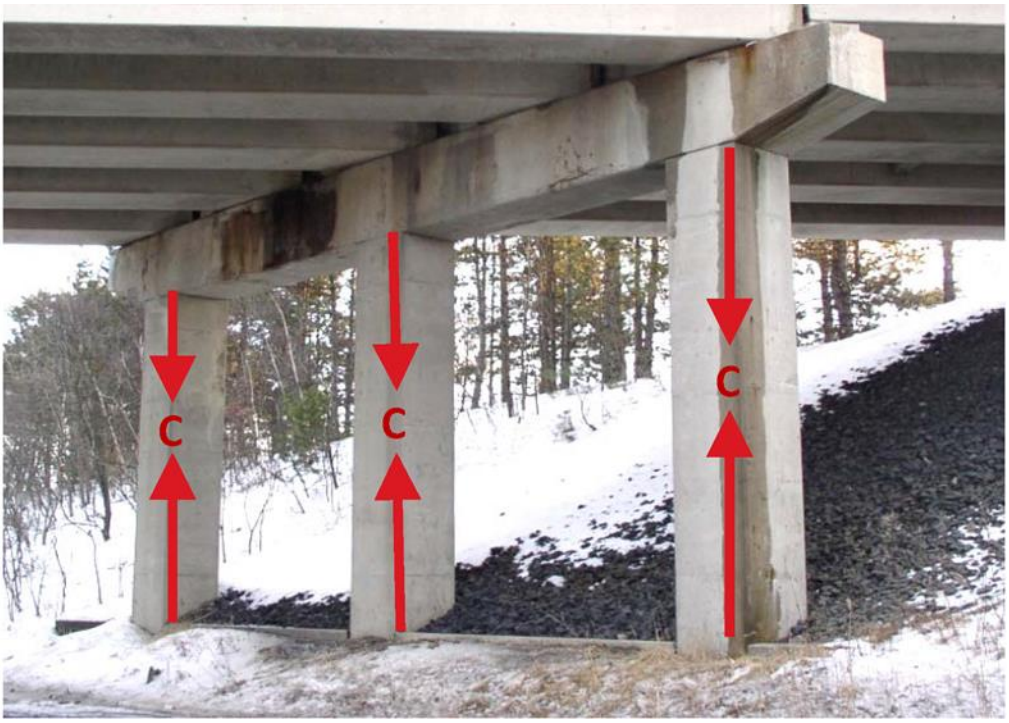

Figure 4.6 Axial Compression in a Bridge

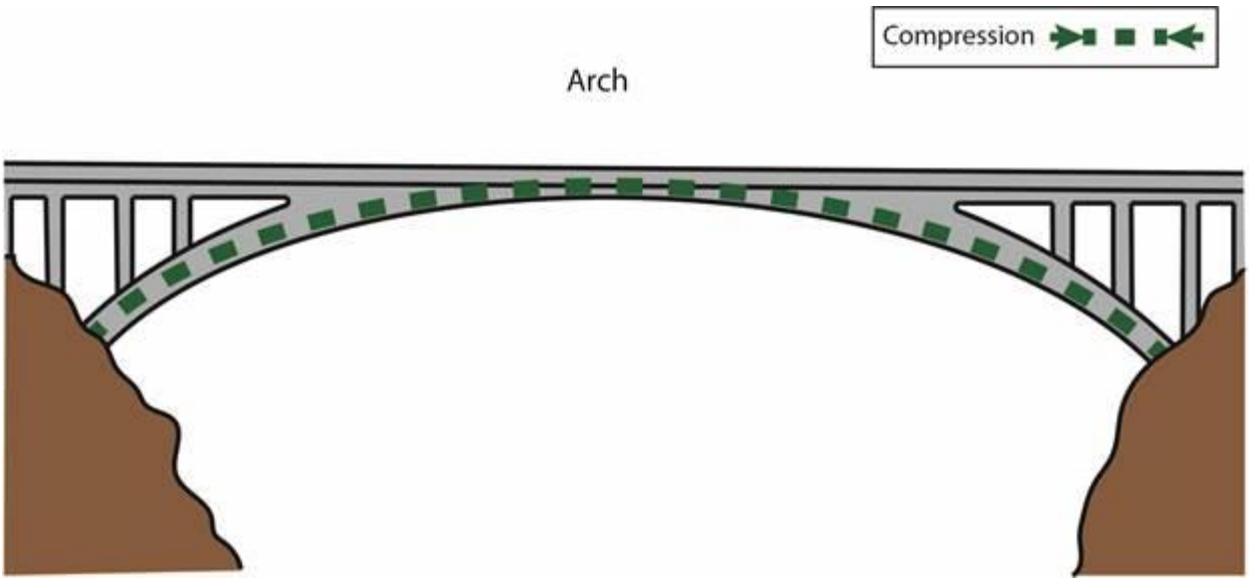

Figure 4.7 Example of Axial Compression in an Arch Bridge

4.2.3 Shear

Shear is a type of force which results from equal but opposite transverse forces that tend to slide one section of a member past another. Figure 4.8 (top) illustrates shearing force. Shear results in both diagonal tension and diagonal compression. Figure 4.8 (bottom) illustrates these forces and the resulting deformation. Where there is diagonal tension and the material is concrete, a crack may form in a direction perpendicular to the force. Similarly for diagonal compression, if the member is a steel plate (e.g., a girder web), the member could buckle and collapse.

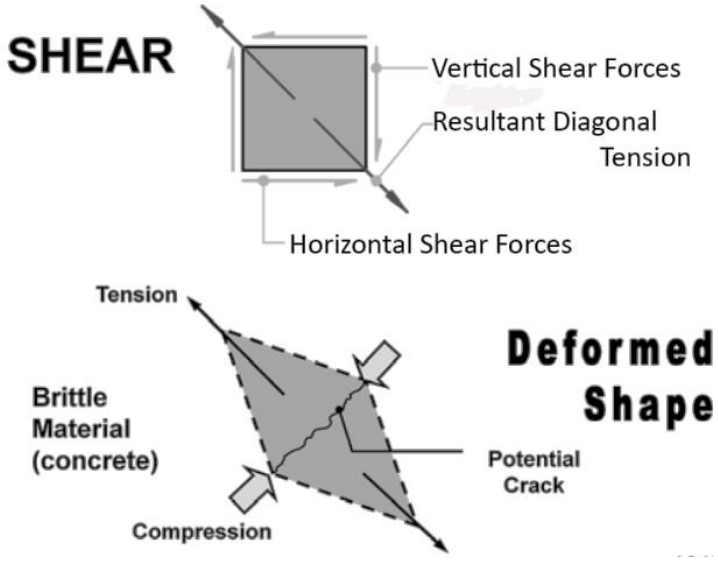

Figure 4.8 Deformation Produced by Shear Forces

A schematic of shear force is shown in Figure 4.9. An example of shear would be the deformation of a deck of playing cards as each card slides passed the one below it when pushed at the top.

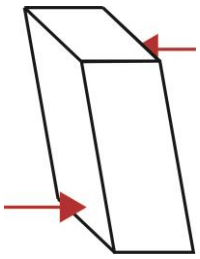

Figure 4.9 Shear Forces

Shear forces are greatest near supports such as a bridge bearing. Figure 4.10 shows a steel bridge girder with a diagonal line to represent a compression path. If the steel girder web is not thick enough, the web could buckle, which would be very serious. For this reason, many times vertical web stiffeners are present near supports to keep the web plate from buckling. Refer to Chapter 3 for more information on stiffeners.

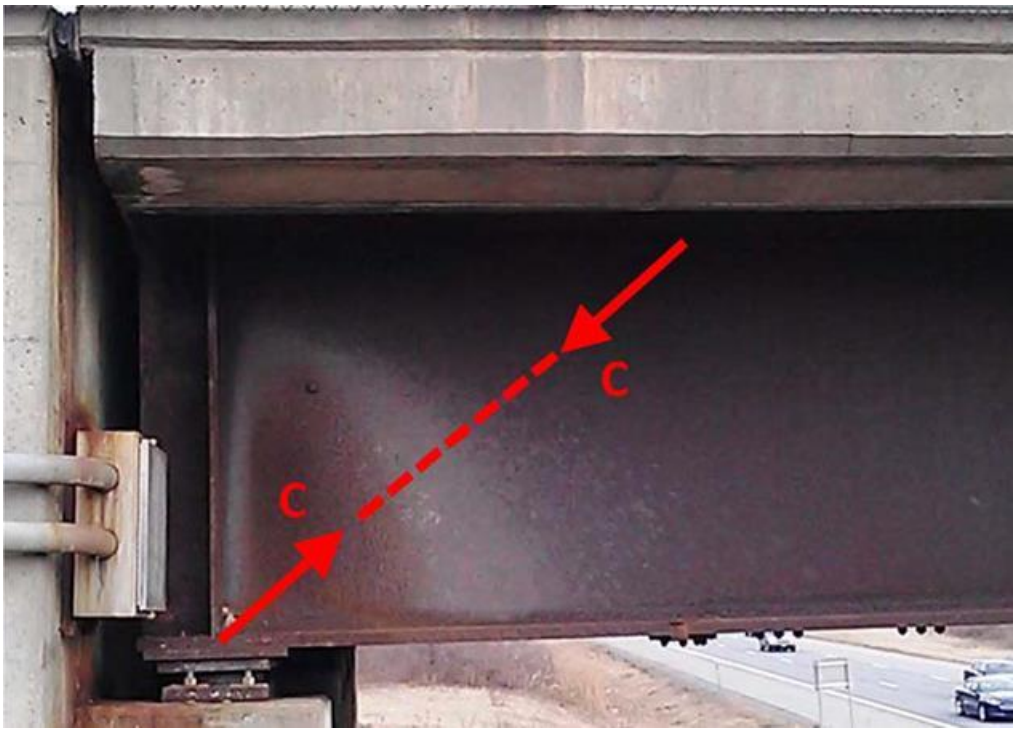

Figure 4.10 Steel Bridge Girder. Diagonal Line Represents a Compression Path.

4.2.4 Flexure

Flexure bends a member and produces a force called a moment, as depicted in Figure 4.11. An example of flexure is a diving board. As the board bends, the top surface is in tension while the bottom surface is in compression. A diving board is a cantilever.

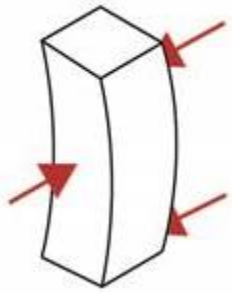

Figure 4.11 Schematic of Flexure in a Member

Moments can produce tension and compression in bridge members depending on the configuration. For example, Figure 4.4 is a suspension bridge. In the middle of the span, the bottom part of the girder is in tension; over the pier, it is in compression. Imagine the bridge wanting to sag in the middle of the span but is unable to do so over the pier because the girder is continuous. Designers determine where the steel girder is in tension and compression and determine flange sizes accordingly, which is why the steel flanges vary in width and thickness on many steel girder bridges. The designer could look at the worst case and make the flanges the same size throughout the bridge, but that would make the bridge very heavy and would be an inefficient design.

4.2.5 Torsion

Torsion is a type of shear force resulting from externally applied moments that tend to twist or rotate the member along its longitudinal axis. Figure 4.12 illustrates this twisting force. A good example is a curved girder bridge, such as shown in Figure 4.13. For a curved bridge, the load is eccentric to a straight line drawn between the two piers, producing a twisting action. For this reason, many curved steel girder bridges have heavy cross frames to counteract this twisting.

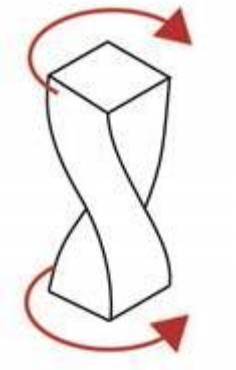

Figure 4.12 Torsion Forces Tend to Twist or Rotate the Member along the Longitudinal Axis

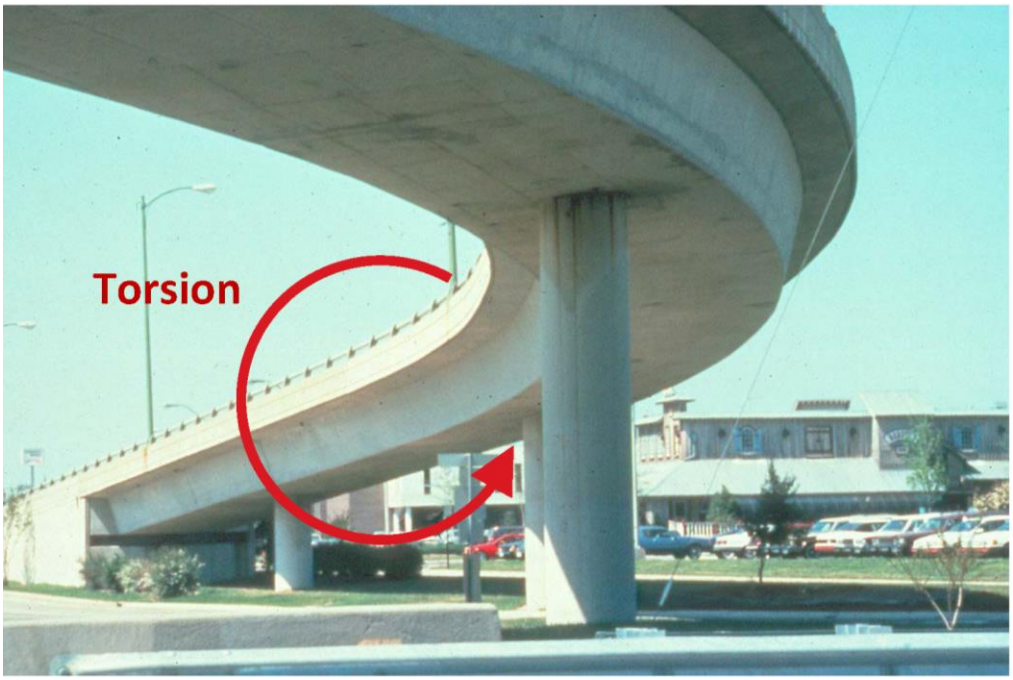

Figure 4.13 Curved Girder Bridge Showing Torsion Forces

4.2.6 Stress

A common term used in bridge design and construction is stress. Stress is defined as the amount of force applied over a given area, as shown in Figure 4.14.

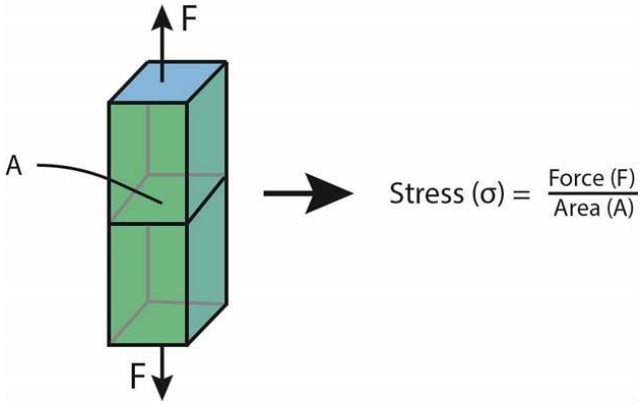

Figure 4.14 Schematic of Applied Stress

Concrete strength is defined as the compression stress at which the material fails. For concrete, this failure strength is about 4,000 pounds per square inch (psi). Steel is typically characterized by its failure stress in tension, or the tensile strength. The steel used in plate girders can have a tensile strength of 36,000 to 50,000 psi. Steel reinforcement typically has a tensile strength of around 60,000 psi. Note that compression strength and tensile strength are material properties, meaning that they don't vary with size of the structure or application.

4.3 Bridge Deck

This section deals with the mechanics of decks, such as load path and composite action.

4.3.1 Load Path

Vehicles apply a vertical load to the deck due to their weight. Gravity applies a pressure at the location of each tire, with the shape of the "footprint" being a rectangle since the rubber tire flattens out under the load. The pressure applied by the tires is transferred through the depth of the deck to the structural members that support the deck. In addition to the vertical loads, there are horizontal vehicle loads. Vehicles braking or accelerating introduce horizontal forces in the deck that must be transferred to the connections between the deck and its supports. Centrifugal forces generated when traffic passes over a curved bridge also produce horizontal loads.

4.3.2 Composite Sections

When concrete decks are used, it is common practice to fix them securely to the steel or concrete beams, so that the deck and beams act in unison. This is called a composite section. In the case of steel beams, shear studs are welded to the top flange before casting the concrete deck. After the deck cures, the concrete is effectively part of the beam. The resulting cross-section is stiffer than the steel beam alone, which reduces deflection due to live and impact loads. A sketch for a composite section is shown in Figure 4.15.

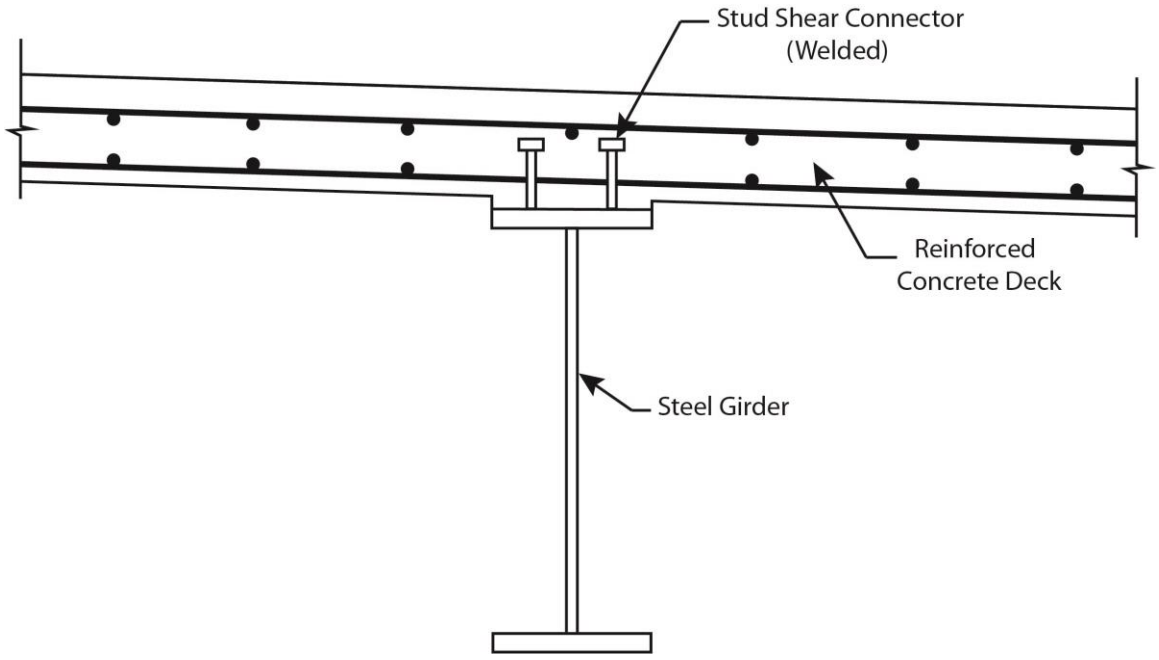

Figure 4.15 Composite Section Comprised of a Concrete Deck on a Steel Girder

4.4 Superstructure

4.4.1 Description

A bridge superstructure can consist of an arch, steel or concrete girders, steel rolled beams, timber beams, trusses, boxes, slabs, cable-supported structures or some other type of structure. The resultant superstructure provides the primary structural support across the stream or road that a bridge is crossing. It literally "bridges" the gap.

In addition to these primary structural members, many bridges also have secondary members as part of the superstructure. These do not provide the primary load path but contribute to the integrity of the bridge in other ways.

4.4.2 Function

A superstructure must meet both strength and serviceability criteria. It must be capable of carrying the loads imposed on it while remaining geometrically stable, and also not deflect excessively. Noticeable deflection could be disconcerting to motorists and could also cause degradation of the structure over time. Preventative maintenance is needed to keep the superstructure in good condition. Without preventive maintenance, deterioration may cause undue stress which may in turn compromise the ability of the superstructure to carry the loads for which it was designed.

Figure 4.16 is an image from a computer model that uses color to represent stress magnitude. This example was produced for a simulation of a deck carrying two heavy dump trucks. The highest stresses in the deck are shown at midspan in red, where the rear axles of two heavy dump trucks were simulated. Most of the girders at midspan are hidden by the deck in this figure, but it is noted that the stresses in the steel girders are maximum at midspan. As indicated by the color, the stresses in the ends of the steel girders are not as great as in the deck under the wheel loads. Deflection of the entire system can be observed.

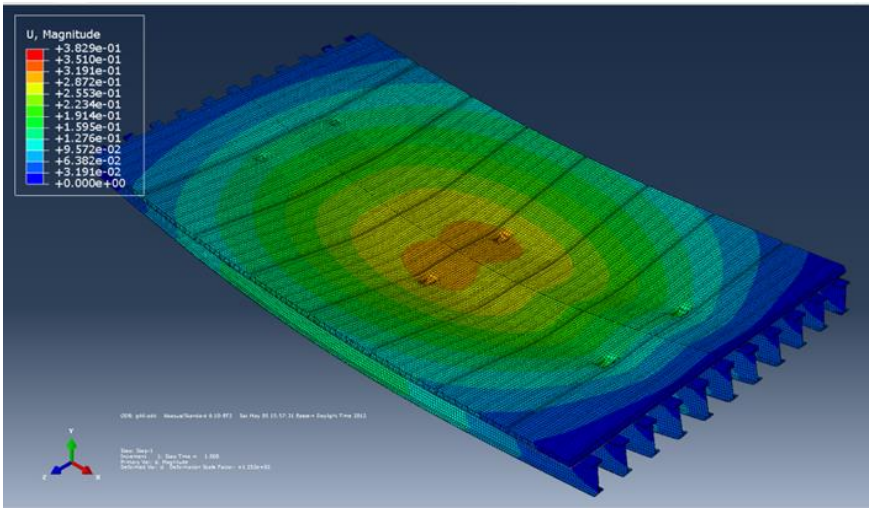

Figure 4.16 Stresses and Deflection in a Simply Supported Multi-Girder Bridge

Figure 4.17 is similar, but shows a one-lane slab bridge. Notice how the slab, which by definition has no girders, produces a different stress pattern. Also, the higher stress areas are at midspan, as will be the case for all simply supported one span bridges.

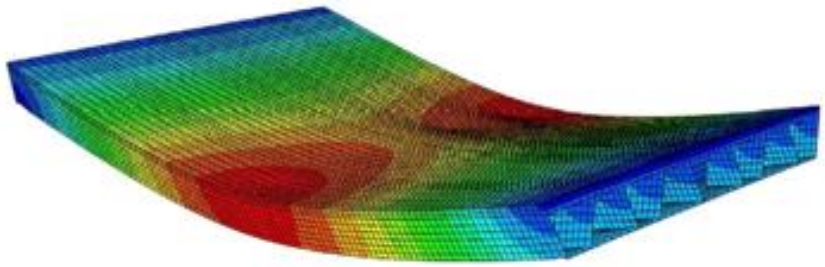

Figure 4.17 Computerized Representation of Stresses and Deflection in a Simply Supported Slab-Type Bridge

The superstructure must be designed to carry its own weight, the weight of the bridge deck, and any other dead, or permanent, load added. Additional permanent weights include the wearing surface, sidewalks, curbing, railings, utilities and lighting. The live, or transient, loads that the superstructure must support consist of vehicle and pedestrian traffic. Other vertical loads include snow and ice. There are also impact loads generated by traffic including braking, acceleration, and centrifugal forces. Other non-static loads are possible, such as loads from wind, stream currents, and earthquakes. Changes in the ambient air temperature, certain weld patterns, steel configurations and connections, corrosion and shrinkage (of wood and concrete) can also cause internal stresses. Design concepts that intentionally put internal stress into a girder include prestressing or post-tensioning of concrete girders. A bridge designer, inspector, or maintainer needs to account for the possibility of any of the above loads. Recent history teaches that there is potential for unexpected extreme events. Tidal surge, tsunamis, fire, or impact from trucks or marine vessels can damage a superstructure; the magnitude of the damage will depend on the location and the exposure of the superstructure to the additional stressors. Bridge loads are listed by type in Table 4.1.

Table 4.1 Typical Loads on Bridges

Dead Loads (Permanent Loads)	Live Loads (Transient Loads)	Dynamic Loads (Non-Static) Or Load Effects from Extreme Events	Internal Loads
• Self-weight of the structure • Wearing surface • Curbing • Sidewalks • Lighting • Utility lines	• Weight of applied moving load of vehicles and pedestrians	• Impact loading from a vehicle • Wind • Earthquakes • Drift against piers • Current • Ice • Impact from being hit by a vehicle or marine vessel	• Thermal stresses from changes in ambient air temperature • Residual stresses from welding • Shrinkage • Pre-stressing • Post-tensioning

Each element in the superstructure must have the strength to resist the applied loads. Dead loads are predictable, live loads less so, and extreme loads, by their nature, are unpredictable. Both the frequency and the magnitude of extreme events can only be estimated.

The function of secondary structural members can be illustrated by considering a multi-girder bridge. These structures usually have cross bracing (or diaphragms) between the girders, as shown in Figure 4.18 left. These are important when the bridge is being built, because they provide stability during steel erection; diaphragms fulfill the same function later if the deck needs to be replaced. Since this cross bracing is in the vertical plane, it also serves the purpose of distributing dead and live loads among the girders. Lateral bracing, sometimes called wind bracing, is another type of secondary structural member (see Figure 4.18 right). This bracing type is placed horizontally between girders to stiffen the structure so it can resist wind loads.

Figure 4.18 Secondary Structural Member Examples in a Superstructure

4.4.3 Load Path

The superstructure (1) carries the weight of the bridge deck and the vehicles on it, and (2) transfers the vertical load to the bearings. The superstructure does this through its individual elements which may include stringers, floorbeams, cross bracings, and other members.

The loads from the deck and the traffic on it can be transferred to the superstructure regardless of the location of the deck in elevation. For instance, a long-span truss may be designed to carry the deck at the elevation of its lower chord, or the truss could be completely under the deck. Since traffic is constantly moving on a bridge, the load passing through the individual elements of a bridge changes as the loads shift. Imagine that there is only one heavy truck on a bridge; the bridge reaction to the weight of the truck changes as the truck moves across the bridge from one side to another. In fact, when the truck first moves onto the bridge, the majority of its weight is applied on just one or two bearings located directly underneath. In that case, the load path is a short one directly through the deck and primary member under it without much load distribution.

When performing maintenance work on elements that are part of the load path, it is imperative that a structural engineer be involved to ensure that all loads can be carried *during* the maintenance project as well as when the project is done. For instance, when removing significant amounts of deteriorated concrete from a column, a maintenance crew will need to have shoring in place to carry the weight of the bridge while repairs are being made. As soon as the weight is taken off a column by shoring under the end of a beam, the load path is changed. Instead of the wheel loads being transferred to deck, superstructure, bearings and substructure columns, the weight is taken up by the temporary shoring, by-passing the bearing and

substructure. This allows for the demolition and removal of concrete in the column so it can be replaced with sound concrete. Once the repair is made and concrete has cured, the shoring can be removed. This restores the original load path.

Figure 4.19 shows a prefabricated steel truss being barged into place prior to erection. Notice that the supports (shown in red) are not at the ends of the span where bridge bearings are usually found. By lifting the bridge at the red supports, the transport team is causing the self-weight of the structure to be transmitted through a different load path (i.e., the red supports) than when placed in service. Loads placed on a structure during construction are frequently different than when in service. Likewise, when a maintenance crew works on a bridge, it is important to remember that the load path in a superstructure might not stay constant. Loads are continually being shifted within the individual elements of the superstructure.

Figure 4.19 Pre-Fabricated Steel Truss on Barges

Any bridge can be mentally broken into its components and elements. For any of these elements, the strength (also referred to as capacity or resistance) of the member must exceed the loads that are going to be placed on the element. Since there is some variability and uncertainty in the strength of the members and the loads applied to them, safety factors are used to ensure that the values used in design are on the conservative side. For instance, if a particular stress cannot be accurately estimated, the assumed stress value will be multiplied by a factor to increase it. In this manner, the assumed stress value is assured to be higher than the stress that the bridge will actually carry. This provides a safety factor, in case the estimated value is low. Likewise, if the strength of a member is uncertain, then the designer will use a lower value so that the computations are conservative enough to cover the discrepancy. Designers call this probabilistic approach “Load and Resistance Factor Design” (LRFD).

Federal bridge inspection standards require that the live load carrying capacity be computed for newly constructed bridges and updated as needed for changes that might have occurred since previous inspections. The load rating can be affected by section loss due to corrosion, damage to a structural member, an increase in weight due to a wearing surface being added, as well as other factors. If the capacity decreases to the point that a bridge cannot carry the truck loads that are driving on it, with a reasonable safety factor, the bridge must be posted with a weight restriction. If this is the case, then it becomes important for operations staff to make sure the regulatory signs are in place and to monitor the types of vehicles using the bridge. If heavy trucks are using the bridge illegally, the police should be contacted to provide enforcement.

4.5 Substructure

4.5.1 Description

An example of a substructure is an abutment. Abutments are the supports at each end of a bridge that support the superstructure. Abutments transfer the loads applied through the bearings and spread them into the footing and foundation below. Piers (or bents) are substructure units used for intermediary support of multiple-span bridges. Either type of substructure unit can be made of steel, concrete, timber, masonry or geosynthetic-reinforced soil systems.

A bridge foundation is the part of the bridge support that is under the substructure. Steel, concrete or timber piles driven into the earth, concrete drilled shafts, or other underground supports, are the foundations for a bridge. Piles can be end-bearing (on rock or firm ground) or friction-bearing, in which case the piles are supported by soil pressures on the sides of the pile.

4.5.2 Function

The purpose of the substructure is to provide a secure support on which the superstructure can rest. Substructures hold back the embankment at the end of the bridge. Since the substructure is in direct contact with the earth, it is subjected to earth pressure, as well as frequent water and ice. The substructure units are required to provide strength, load distribution, and protect against settlement, tipping, overturning, and sliding at the base. Settlement or distortion of a substructure can induce unanticipated stresses in the superstructure. Scour, the loss of earth under the substructure in a river or marine environment, is a very important issue that is discussed in detail in Section 4.8.3. Erosion at the abutments can also jeopardize the integrity of the structure or render it unusable by cutting off access.

While the substructure needs to carry the loads imposed on it, it is just as important for it to maintain stability of the structure. For the substructure to function as intended, the substructure must bear securely on the soil or rock underneath it. Settlement, scour, erosion, and impact from a vessel or vehicle each have the potential to initiate instability and, if severe enough, they can cause collapse. Ground motion during an earthquake can also cause such instability.

4.5.3 Load Path

Since gravity pulls objects downward, the substructure must bear the weight of anything above it, as shown in the schematic in Figure 4.20. The schematic shows load being transferred from a truck through the deck into the girders, abutment and the ground. The substructure carries the weight of any vehicles that are on the bridge, the weight of the deck, the weight of the superstructure and bearings, as well as the effect of other loads.

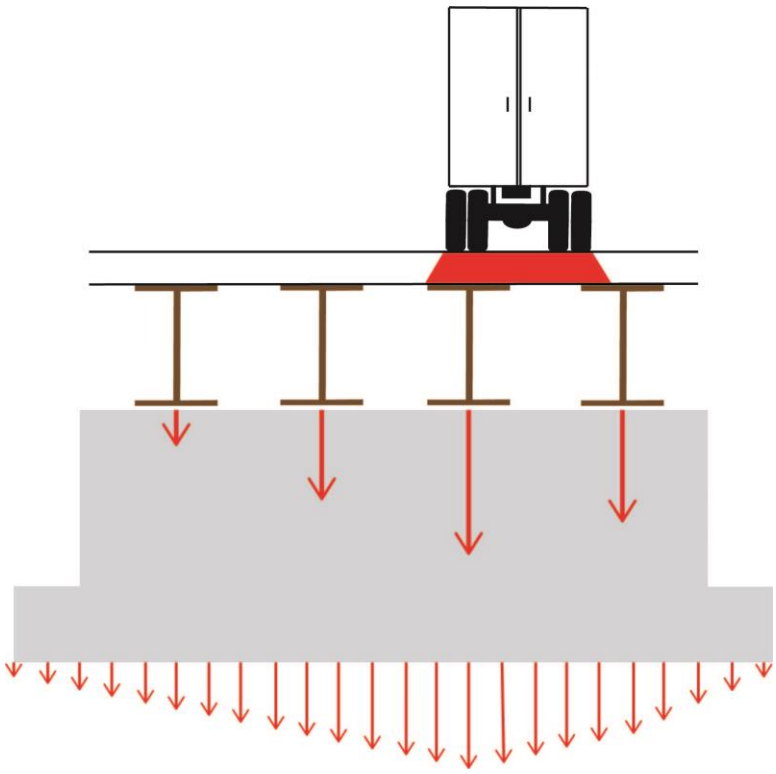

Figure 4.20 Bridge Cross-Section Illustrating Load Transfer

4.6 Culverts

4.6.1 Description

There are many types of structures that enable one to overcome an obstacle such as a stream, or road. According to the National Bridge Inventory (NBI), such a structure is classified as a bridge if it spans more than 20 feet. Although some smaller structures look just like a multi-girder bridge, they are considered culverts if less than 20 feet. The span is measured from one retaining wall to the other along the direction of traffic, directly under the center of the road. Examples of several types of culverts are provided in Section 3.1.5.

There are also many culverts over 20 feet. These are concrete box culverts, multi-plate pipe arches, or a series of smaller, closely-spaced pipes whose total span exceeds 20 feet. Approximately one of every five NBI structures in the country is a culvert (Barker and Puckett, 2007).

4.6.2 Function

Culverts are used to convey small streams under roadways, but also can be used for a street, pedestrian walkway, or even a cattle-pass under a highway embankment. Culverts can be cast-in-place or precast concrete, steel or aluminum pipe or plate arch, timber, or even FRP composite material. Culverts can have a bottom or not, depending on use and structure type. Culverts are included in this document because of their similarity to bridge structures. Much of the information presented is applicable to both.

4.6.3 Load Path

Although there are a variety of types of culverts, a general description of the load path may be helpful. The many varieties of culverts fall into two general categories: buried and unburied. A buried culvert has traffic load applied through an earth fill above it that is at least one foot thick. The required depth of earth fill varies according to the span. A good understanding of the soil-structure stress interaction is necessary since the surrounding soil transfers wheel loads to the structure. Traffic is maintained on a pavement, and the tires apply pressure to the pavement and subsequently the earth above the structure. Assuming the depth of earth cover is sufficient, this distributed earth pressure does a good job of transferring the dead and live loads to the top of the culvert. Depending on the type of structure, the loads can be transferred through the structure into the ground like an arch. Or, in the case of box with a horizontal roof, the roof carries loads to the vertical supports, using the roof as a means of transferring the loads, since it acts much like a slab bridge would.

If a culvert is not buried, traffic drives directly on it or on a wearing surface with no fill above the culvert. The mechanics of culvert load transfer are similar to that described in Section 4.4.3 Load Path, but dramatically different from a buried structure and so require different analysis methods.

4.7 Redundancy and Fracture Mechanics

Bridges that have more than one path for carrying loads are said to be redundant. Historically, limitations in computational capacity and the desire for efficient structures for economic reasons led to the construction of non-redundant bridge systems. Three types of redundancy are discussed in the following sections.

4.7.1 Load Path Redundancy

Structures can be protected from catastrophic failure by providing load path redundancy. Load path redundancy means having more than one load path. Truss bridges are considered non-redundant because there are only two trusses (as shown in Figures 3.53 and 3.54). If one fails, the entire span can collapse because there is no alternate load path. An example of this situation is the I-35W bridge that collapsed in Minneapolis in August 2007. Another example of a non-redundant bridge is the thru-girder bridge shown in Figure 3.51 and Figure 4.21 (left). A multi-girder bridge has numerous load paths, as shown in Figure 3.49 and Figure 4.21 (right). When many primary members are present, if one girder on a bridge fails, load can then redistribute among the other girders.

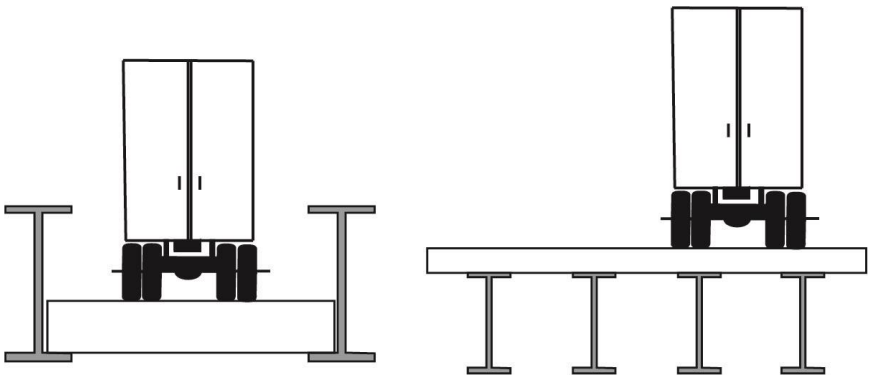

Figure 4.21 Non-Redundant Load Path Bridge (left) vs. Redundant Load Path Bridge (right)4.7.2 Internal Redundancy

If a primary structural member is comprised of multiple elements, due to the method used for its fabrication, the member may have the ability to redistribute loads should one element crack or fail by some other means. An example is a built up riveted girder, because a defect in one of the internal elements cannot propagate to another and lead to the failure of the entire main member. Cracks can easily propagate through welds, so welded plate girders are not internally redundant. Examples of internally redundant and internally non-redundant girders are presented in Figure 4.22. In Figure 4.22 left, the riveted built-up girder is internally redundant, whereas the welded plate girder in Figure 4.22 right is internally non-redundant.

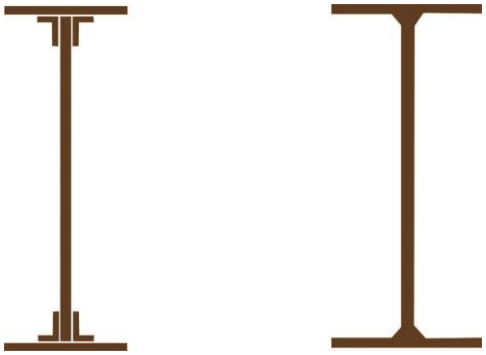

Figure 4.22 Internally Redundant Girder (left) vs. Internally Non-Redundant Girder (right)

4.7.3 Structural Redundancy

When a bridge consists of more than one span, a pier or bent is used as an intermediate support. In the past, it was common to construct these bridges as independent (i.e., simple) spans because analysis methods available at the time made it difficult to design other types of structures. A superstructure is said to have continuity if the primary structural members are continuous over the piers. Figure 4.23 shows a pier with a gap between the two girders and a bearing under each. This is an example of a bridge that is *not* structurally redundant. A structurally redundant bridge would have one beam continuous over the pier, or possibly two beams bolted together as part of a retrofit project to make them continuous.

Figure 4.23 Structurally Non-redundant Multiple Span Bridge

Continuity is desirable for several reasons. Continuity is more efficient structurally as it eliminates a joint over the pier that could leak and cause deterioration of the substructure. Continuity also helps to ensure stability of the structure if it is subjected to an earthquake, storm surge, or if the support of the pier is lost due to a truck impact or scour. Imagine a large truck hitting the pier and tilting it over a foot. The girder could easily fall off the pier ledge. Having the girders continuous over the pier would prevent this. Some older bridges can be made "continuous for live load," by casting a concrete deck continuously over the gap between the ends of two adjacent spans to eliminate the pier joint.

Figure 4.24 shows a discontinuity at each pier in the left figure, indicating that the bridge is made up of multiple simple spans. The superstructure in the right figure is continuous over all piers, providing structural redundancy for the interior spans. The end spans are not redundant because there is no continuity at the end of the bridge. If one pier is lost due to scour or vehicular impact, the superstructure would not be in imminent danger of collapse.

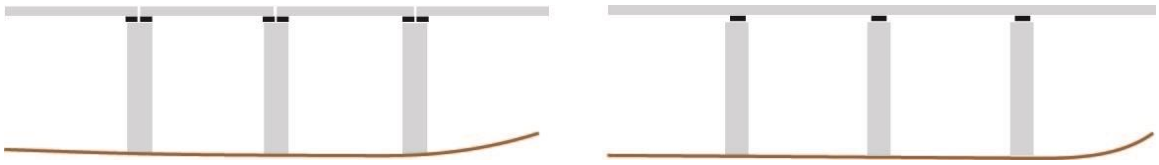

Figure 4.24 Structurally Non-Redundant Bridge vs. Structurally Redundant Bridge

4.7.2 Simple Span

A simple span bridge is the easiest bridge type for an engineer to design, an example of which was shown in Figure 4.23. Many states liked the simplicity of the simple span bridge, in that if one component got damaged then it could easily be removed and replaced. However, simple span bridges have discontinuities in the superstructure at every support, and therefore have many joints. This is undesirable from a maintenance standpoint. Joints cause problems. Joints leak and allow salts and other contaminants to accumulate on the substructure below.

4.7.3 Continuous Span

Continuous span bridges are popular because deck joints can be reduced or eliminated, as shown in Figure 4.25. However, engineers must consider this continuity in their design. For example, a truck in one span can cause the adjacent span to deflect upwards due to the continuity of the girders.

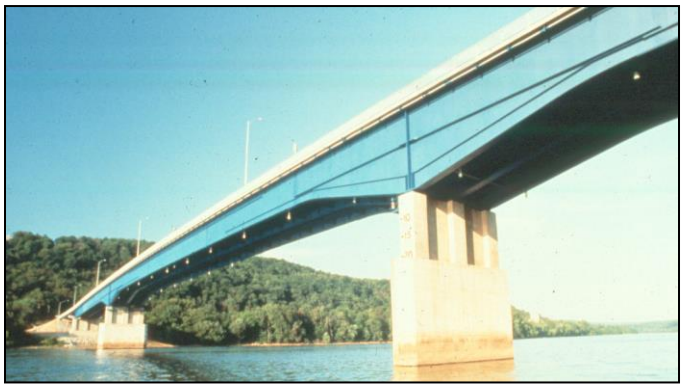

Figure 4.25 Continuous Span Steel Bridge

4.7.4 Cantilever Span

Cantilever span bridges not only have bearings located located over the piers, but they some type of bearing assembly on the anchor spans supporting the cantilever spans, as shown in Figure 4.26. In some applications with steel girders, pins and hangers are used instead of bearings to connect the girders together.

The joints located with the span are the most critical expansion joints for bridge maintenance. The deterioration of girder ends and bearings that is common below expansion joints can cause a serious lack of support for the suspended span if the bearings or pin and hanger assemblies fail.

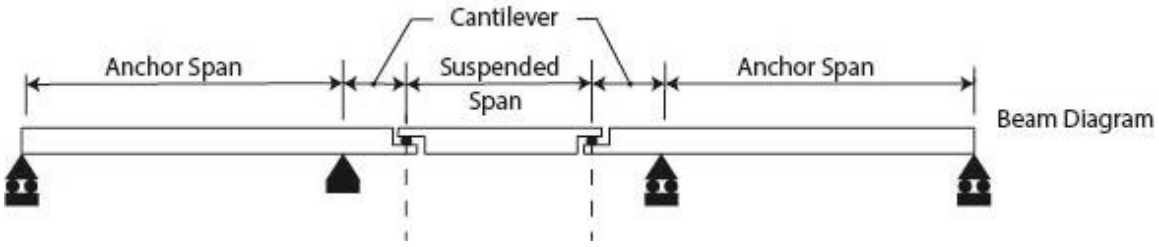

Figure 4.26 Cantilever Span

4.7.5 Fracture Mechanics

A maintenance worker should sound an alarm when a crack is found in a metal member. The rate that a crack propagates depends on numerous factors, but in certain instances a crack can occur suddenly and cause total collapse of a structure, especially if the structure is non-redundant. Stress levels, load cycles, temperature, and fabrication details all play a role in how fast a crack grows. The agency's bridge inspection office should be contacted whenever a crack is discovered in a primary steel bridge member.

Fracture toughness is a material property used to describe the ability of a metal to absorb energy without fracturing (or cracking). In steel, fracture toughness is commonly measured using a Charpy V-notch test. Certain materials, such as wrought iron, are especially susceptible to cracking. Because wrought iron exhibits low fracture toughness, even a small crack initiated by a material flaw can propagate and become a serious issue after a wrought iron element is placed in service. Even in modern steels, weld defects and certain weld details can initiate cracking. If a steel structural member is subjected to tensile and compressive stresses above a certain threshold range, fatigue cracks (i.e., cracks due to repeated cycling at stress significantly

less than the yield strength) will eventually develop due to stress cycling. Engineers have methods to estimate "remaining fatigue life" of a structure, but such methods rely on estimates of past truck traffic, particularly the truck weights and number of vehicles. Out-of-plane bending, typically where a diaphragm connects to a girder, can cause distortion-induced fatigue. Even small welds can cause major problems. A tack weld in a tension zone of a primary member can lead to cracking in primary members. For existing structures, anything that is attached to any part of the primary member with welds should be monitored for evidence of cracking. No welding should be undertaken without the involvement of a knowledgeable professional engineer.

It is important to be able to identify certain weld details that are known to be fatigue prone. AASHTO's Manual for Bridge Evaluation contains detailed guidance. Examples of fatigue prone details are transverse cover plate end welds, the intersection of multiple welds, and spot welds.

4.8 Waterway and River Mechanics

Before any work is conducted in a waterway, it is important to apply for any required permits from regulatory agencies such as the Corps of Engineers and the state environmental agency. Often isolated site work near a bridge can have unanticipated negative impact elsewhere along the river.

4.8.1 Purpose and Function of Channel Protection Features

Moving water can do serious damage to bridge structures. In fact, about sixty percent of U.S. bridge failures are due to hydraulic forces (http://www.fhwa.dot.gov/advancedresearch/pubs/10034/10034.pdf). In recent years, most DOT agencies have adopted a policy of installing piles or other scour protection measures under any substructure that is situated in water. Since flood waters are often turbulent and have high velocity, it is important that the river banks and substructures be able to withstand stresses imposed by such conditions. Vegetated banks are less susceptible to erosion. When properly designed, large stone (riprap) or paving blocks can be used to line channels and armor the bank close to the bridge. Inspection and maintenance of channel protection is important.

4.8.2 Basic Hydraulics

When a bridge is designed, it must be sized according to the volume of water that is expected to pass under it. This is done with rainfall intensity maps or equations for a given site, using a drainage area taken off of topographic maps. Computer programs originally developed by the U.S. Army Corps of Engineers (USACE) are used with survey data upstream and downstream from a bridge site, and the elevation of the design flood is determined. Usually, additional freeboard (the space between design high water and the bottom of the bridge) is provided so that trees, branches, and other debris carried by flood waters can pass under the bridge.

If flood waters get high enough to reach the bridge, the hydraulic pressure can cause the superstructure to move laterally or even uplift. This also affects how the water passes under the bridges. This pressure flow condition causes the water to speed up as it passes through the constricted opening, exacerbating scour at the site.

4.8.3 Scour

Scour is the term used to describe the loss of stream bed material due to hydraulic action. An example of severe scour is shown in Figure 4.27. A bridge is considered scour-critical if there is reasonable chance that scour can undermine the bridge, for instance if a pier in the water does not have any piles, the piles it has are known to be too short, or if the depth of the foundation is unknown. Scour is described in the following sections of this manual as one of three types (i.e., general, contraction, or local).

Figure 4.27 Example of Severe Scour (Courtesy New Hampshire DOT)

4.8.3.1 General Scour

General scour is a lowering of the streambed a distance up and downstream of the bridge. It is not caused by the bridge but by the natural tendency of the entire stream bed to decline in elevation due to soil particles moving downstream, especially during time of high water because fast moving water can easily pick up larger stones, as well as soil particles. During flood stage, a stream may be turbid because it is carrying much more sediment than during normal, non-flood conditions. General scour occurs because, over time, water picks up sand, stone and rocks and carries them downstream. Even a stream on bedrock can erode given enough time. Degradation can be exacerbated by gravel mining downstream of the bridge; a hole is created that the stream will naturally fill. Material is carried downstream and deposited in the created hole. The elevation of a streambed can also rise due to aggradation. The elevation and alignment of stream channels can change significantly over time, so bridge inspectors use drop-line readings from a bridge to measure the relative elevation of the bed and document the channel profile in the inspection reports. A schematic of general scour is shown in Figure 4.28 (top).

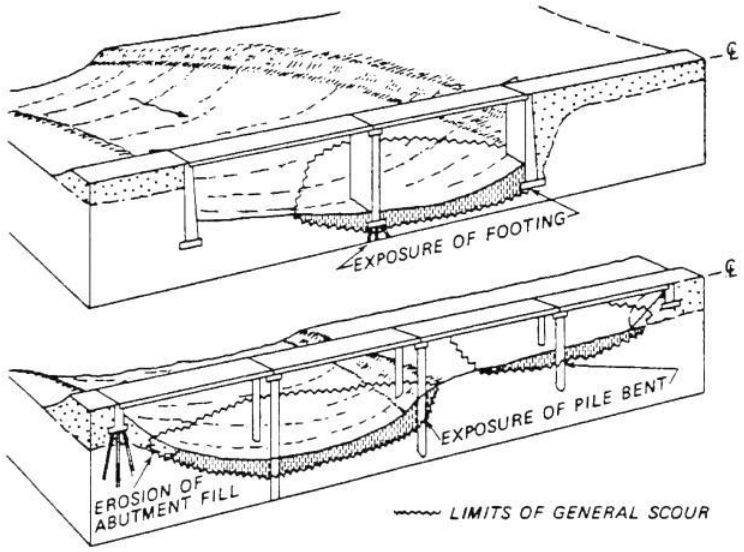

Figure 4.28 General Scour Schematic (top). Contraction Scour Schematic (bottom).

4.8.3.2 Contraction Scour

Contraction occurs when the opening under a bridge is too narrow for the flood waters running under it. One of the basic principles of hydraulics is that flowing water will speed up as it passes through a restricted area. Fast moving water picks up particles from the bottom of the stream, then causing scour. A schematic of contraction scour is shown in Figure 4.28 (bottom).

4.8.3.3 Local Scour

Local scour can occur at a pier that is located in the water. A good design will avoid piers in streams, but it is not always possible due to the expense associated with longer spans. If a pier on a new bridge must be a stream, it will be situated on piles that provide support in case scour does occur. If an existing bridge pier does not have piles under it, heavy stone can help protect it or sheet piling can be added.

Local scour can create very deep holes, over 20 feet in some cases. As flowing water hits the nose of a pier, it splits and flows around the pier stem and is forced in a downward direction (see Figure 4.29). The downward flow digs into the stream bed and makes the stone, gravel and sand waterborne, where it can be swept away in the current. Bridge skew, water velocity, size and shape of the stones in the riverbed all contribute to the severity of the resultant scour.

Some scour-critical bridges have been equipped with scour monitoring devices. Scour monitoring devices should be checked regularly to ensure proper operation.

The upstream nosing of piers in a river is usually rounded or V-shaped. The purpose of this geometry is to facilitate the flow of water around the pier. From a maintenance perspective, every effort should be made to keep this part of the pier clear of debris such as branches and logs. Debris and ice that catch on the nosing of a pier disrupt the smooth flow around the pier, causing turbulence and eddies. The disturbed flow results in roily water and scour.

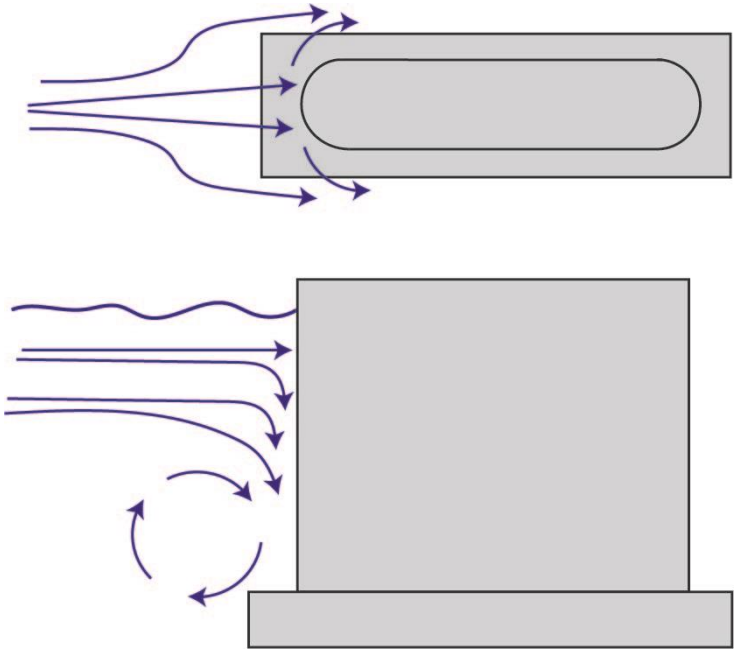

Figure 4.29 Local Scour at a Pier. Plan View (top) and Elevation View (bottom).

4.8.4 Channel and Embankment Erosion

Channel and embankment erosion are similar to scour, but occur to an embankment or soil that is normally dry. Heavy rains can initiate erosion of vegetation; without vegetation, the soil becomes exposed which then allows deterioration of the stream bank. Erosion can be a problem at the ends of bridges where the bridge has inadequate or poorly maintained drainage details.

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Chapter 5 - Concrete Basics

5.1 Concrete Overview

Concrete is a composite material that uses cement to bind together crushed stone, rock, and sand (i.e., aggregate). Cement acts as a binder filling the space among the aggregate particles and glues them together. The exact proportions of aggregate, cement, and water vary depending on the type of concrete being made. The aggregate and cement are mixed thoroughly with water, which starts the chemical reaction causing the cement to harden and set. This chemical reaction is called hydration.

Quality concrete should have the following characteristics:

- It should be constructible. This means concrete mix should be workable, and placed and consolidated with no difficulties. It should flow sluggishly without segregation of the ingredients.
- It should gain the required strength after it is cured.
- It should be durable. This means during the service life, concrete should not develop premature deterioration.

Certain properties may be required, depending on the type of environment, to resist the following types of deterioration:

- Surface cracking (shrinkage cracks) caused by improper curing
- Surface scaling caused by wetting and repeated freeze/thaw action
- Surface scaling caused by deicing salts
- Chemical reaction caused by sulfate action or certain types of aggregate
- Abrasion caused by traffic
- Penetration of water and deicing salts and subsequent corrosion of the reinforcing steel

The most popular type of concrete is hydraulic cement concrete. Polymer concrete is very popular for bridge maintenance repairs, but there are also many other types of concrete including high performance, self-consolidating and lightweight which will be explained in the following sections.

One of the more confusing aspects of concrete is the amount of terminology. Simple definitions of some of the more confusing terms used in this chapter are provided below.

Pozzolan: A material that can be substituted for cement. A pozzolan is a siliceous or siliceous and aluminous material which, in itself, possesses little or no cementitious value, but which will, in finely divided form and in the presence of water, react chemically with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties. Examples are slag, silica fume, or fly ash.

Blended Cement: A cement that also contains pozzolans (supplementary cementing materials) such as slag, silica fume, or fly ash.

Admixture: A chemical material that is added to the concrete mix to enhance its properties.

Aggregate: The material in concrete that is rock-like

Plastic Concrete: Concrete state before it hardens, it can flow

Air-entraining: The ability to insert tiny air bubbles into concrete

High early strength: The ability for concrete to gain strength quickly

5.2 Components and Properties of Hydraulic Cement Concrete

5.2.1 Cement

Cement is a very fine powder made of limestone calcium, silicon, iron, and aluminum, among other ingredients, which absorbs water and acts as a binder to hold the aggregates together. The raw ingredients of cement are heated in large kilns to about 2,700 °F to form a product known as clinkers, which roughly resemble marbles. These are ground into a powder and gypsum is added, creating the gray flour-like substance known as cement. When water is added to cement, it triggers a chemical reaction called hydration which allows it to harden. Cement typically makes up from 10 percent to 15 percent of the total mass of concrete.

When to Call the Engineer

Concrete mix design and selection should be performed by an engineer or technician properly trained in concrete mix design and knowledgeable with the exposure environment, placement conditions and desired properties.

5.2.1.1 Types of Cements

There are many different types of cement, but the type most commonly used in construction is Portland cement. The word “Portland” is not a proprietary term; it indicates the type of cement much like the word “stainless” is used to describe a type of steel. Portland cement is a type of hydraulic cement. Different types of Portland cement are manufactured to meet different physical and chemical requirements for specific purposes, such as durability and high-early strength (i.e., quick setting, typically achieved by addition of an accelerating admixture). Air entraining agents are sometimes added to intentionally create tiny air bubbles in the concrete. The addition of tiny air bubbles can increase the durability of the hardened concrete and also increase workability of the concrete while in a plastic state.

Eight types of cement and a brief description of their uses are listed in Table 5.1.

*Table 5.1 Portland Cement Types and Their Uses**

Cement Type	Use
I	General purpose cement, when there are no extenuating conditions. Cements that simultaneously meet requirements of Type I and Type II are also widely available.
II	Aids in providing moderate resistance to sulfate attack. Type II low alkali (total alkali as Na_2O < 0.6 percent) is often specified in regions where aggregates susceptible to alkali-silica reactivity are employed.
III	When high-early strength is required (high strength at early time period).
IV	When a low heat of hydration is desired (in massive structures). Type IV cements are only available on special request.
V	When high sulfate resistance is required. These cements are in limited production and not widely available.
IA	A Type I cement containing an integral air-entraining agent. These cements are in limited production and not widely available.
IIA	A Type II cement containing an integral air-entraining agent. These cements are in limited production and not widely available.
IIIA	A Type III cement containing an integral air-entraining agent. These cements are in limited production and not widely available.

* American Society for Testing and Materials (ASTM) Standard Specification for Portland Cement: ASTM C150

More than 92 percent of Portland cement produced in the United States is Type I and II (or Type I/II); Type III accounts for about 3.5 percent of cement production (U.S. Department of the Interior, 1989). Type IV cement is only available on special request, and Type V may also be difficult to obtain (less than 0.5 percent of production).

Although Type IA, IIA, and IIIA (air entraining cements) are available as options, concrete producers prefer to use an air entraining admixture during concrete production, where they can get better control in obtaining the desired air content. In general, air entraining agent is added at the plant and if necessary adjusted in the field. However, caution is advised as the quality control is poor, particularly when no means of measuring the air content of fresh concrete is available. Entrained air content can be determined in accordance with the requirements of ASTM standards, i.e., ASTM C231 or ASTM C173.

If a given type of cement is not available, comparable results can frequently be obtained by using modifications of available types. High-early strength concrete, for example, can be made by using a higher content of Type I cement when Type III cement is not available (National

Materials Advisory Board, 1987), or by using admixtures such as chemical accelerators or High-Range Water Reducers (HRWR). The availability of Portland cements will be affected for years to come by energy and pollution requirements. In fact, the increased attention to pollution abatement and energy conservation has already greatly influenced the cement industry, especially in the production of low-alkali cements.

Three types of cement are commonly used in the repair of highway structures.

- Type I, or "Normal" cement, is general purpose cement.
- Type II cement is used when concrete is to resist moderate sulfate attack from sources such as ground water. Also, since Type II hydrates at a slower rate than Type I, there is generally less heat generated. As a result, it may be used in structures of considerable mass, such as large piers, heavy abutments, and heavy retaining walls to reduce rise in the temperature of the concrete mass. If the temperature rise is not controlled, concrete can crack upon cooling since it will shrink substantially. For the same reason, Type II cement is also a desirable cement to be used for concrete to be placed in warm weather.
- Type III is a high-early-strength cement which develops higher strength at an earlier age. This is the desirable cement to be used for concrete patches where early use of the roadway is important. However, unlike Type II cement, Type III will generate more heat of hydration than Type I. Therefore the use of Type III cement is not desirable in large structures and in warm weather construction, since thermal shrinkage cracks can occur after curing.

5.2.1.2 Physical Properties of Cement

ASTM has specified certain physical requirements for each type of cement. These properties include 1) fineness, 2) soundness, 3) consistency, 4) setting time, 5) compressive strength, 6) heat of hydration, 7) specific gravity, and 8) loss of ignition. Each one of these properties has an influence on the performance of cement in concrete. The fineness of the cement, for example, affects the rate of hydration. Greater fineness increases the surface available for hydration, causing greater early strength and more rapid generation of heat (the fineness of Type III is higher than that of Type I cement) (U.S. Department of Transportation, 1990).

5.2.2 Aggregates

Aggregates make up more than 60 percent of a concrete mix — and up to 80 percent in some cases. Aggregate is also less expensive than cement, so a higher percentage can lower the cost. Thus, the concrete mix designer can often save money by selecting the maximum aggregate size allowable. Using larger coarse aggregate typically lowers the cost of a concrete mix by reducing cement requirements, the most costly ingredient. Less cement (within reasonable limits for durability) will mean less water if the water-cement (w/c) ratio is kept constant. A lower water content will reduce the potential for shrinkage and for cracking associated with restrained volume change. Generally speaking, a good aggregate has a combination of rocks of many different sizes, graded, with a specific average and maximum size; the aggregate must be clean and durable, and should not contain clay or other minerals that can absorb water.

The high rock content of concrete makes concrete extremely durable, and it often is used in roadways, bridges, and airport runways. The ingredients in both concrete and cement are

among the most abundant on earth, and both can be recycled. Cement production does require a large amount of energy, however, because of the high temperatures required.

The characteristics of aggregates strongly influence the properties of the concrete. To produce high quality concrete, the aggregate should consist of clean, hard, strong, and durable particles free of chemicals, coatings of clay, or any other fine materials that may affect the hydration and the final bond of the cement paste. Weak, pliable, or laminated aggregate particles are undesirable. Aggregates containing natural shale or shale particles, soft and porous particles, and certain types of chert (a form of microcrystallite quartz) should be especially avoided since they have poor resistances to weathering.

Generally, the following characteristics are expected of aggregate:

- Freeze-Thaw Resistance
- Chemical Compatibility with Cement
- Particle Shape and Surface Texture
- Gradation and Size

Each of these characteristics is further described below.

In exposed concrete, aggregate should be resistant to repeated cycles of freezing and thawing. In the presence of absorbed water, repeated cycles of freezing and thawing can deteriorate the aggregate and reduce the strength of concrete with time. Standard tests, such as American Association of State Highway and Transportation Officials (AASHTO) Designation TP17, are available to determine the freeze-thaw resistance of aggregate.

Aggregate should chemically be compatible with cement. Some aggregate may react chemically with cement and cause expansion of concrete and subsequent severe cracking of concrete. This is called "alkali-aggregate" reaction, and the aggregate is called "alkali-reactive" aggregate. Tests are available to identify alkali-reactive aggregates. If alkali-reactive aggregate is the only aggregate available, cements with certain chemical composition or admixtures may be selected to prevent the alkali-aggregate reaction.

These characteristics of an aggregate mainly affect the properties of the plastic or formable (prior to hardened) concrete. Aggregates with rough texture or flat and elongated particles require more water to produce workable concrete than round or cubical aggregates. In turn this can result in a higher water-cement ratio and lower strength. However, aggregates with rough and irregular shapes tend to lock together to produce a stronger matrix than smooth rounded shapes.

Generally, an aggregate gradation with minimum void content is desirable, since the cement paste requirement for concrete increases as the void content in the aggregate is increased. Aggregate with uniform particle size increases void content. To provide the minimum void content, coarse and fine aggregate are combined. Coarse aggregates are particles that generally range between 3/8 and 1.5 inches in diameter, and fine aggregates generally consist of natural sand or crushed stone with most particles passing through a 3/8-inch sieve. Gradation limits for coarse and fine aggregate are specified to ensure minimum void content.

The maximum size of coarse aggregate in the mix depends on the size of the concrete member or size of repair, and on the spacing of the reinforcing steel. The maximum size of coarse aggregate should be small enough to place the mix in the form and in the spaces between the

reinforcing steel with no difficulties. Specifications give the maximum size of aggregate on the basis of the minimum dimension of the form and clear space between the reinforcing bars. Also, the smaller the maximum size of aggregate, the greater will be the amount of mixing water required to produce the same workability. Therefore, it is advantageous to use the largest practicable maximum size of coarse aggregate.

5.2.3 Water

Certain levels of chemicals and impurities in mix water can reduce the strength of concrete and cause its deterioration in time. Water that is suitable for drinking is generally preferred as mixing water for making or curing concrete. Conversely, water that is suitable for making concrete may not necessarily be fit for drinking. Therefore, if it comes from a source other than drinking water, the water should be tested to determine that it has the right chemical makeup for making concrete. Examples of components of concern include chlorides (rebar corrosion concern) or alkali carbonates and bicarbonates sometimes occurring in some natural mineral water (alkali-silica reaction concern).

5.2.4 Admixtures

Admixtures are ingredients other than water, aggregates, hydraulic cement, and fibers (sometimes added to reinforce concrete) that are added to the concrete batch immediately before or during mixing. A proper use of admixtures offers certain beneficial effects to concrete, including improved quality, acceleration or retardation of setting time, enhanced frost-freeze-thaw and sulfate resistance, control of strength development, improved workability, and enhanced finishing. It is estimated that 80 percent of concrete produced in North America these days contains one or more types of admixtures. According to a survey by the National Ready Mix Concrete Association, 39 percent of all ready-mixed concrete producers use fly ash, and at least 70 percent of produced concrete contains a water-reducer admixture.

Admixtures vary widely in chemical composition, and many perform more than one function. Two basic types of admixtures are available: chemical and mineral. Mineral admixtures are also called Cementitious Admixtures. Chemical admixtures are generally added to concrete in very small amounts while mineral admixtures are added in larger amounts. All admixtures to be used in concrete construction should meet specifications; tests should be made to evaluate how the admixture will affect the properties of the concrete to be made with the specified job materials, under the anticipated ambient conditions, and by the anticipated construction procedures.

Admixtures are added to concrete with water before mixing the concrete, or during mixing cement, aggregate, and water. Admixtures are generally classified as follows:

- Air-Entraining Admixture
- Water-Reducing Admixture
- High-Range-Water-Reducing Admixture (Superplasticizer)
- Retarding Admixture
- Accelerating Admixture
- Cementitious Admixtures
- Corrosion Inhibitor Admixtures

Admixtures may be used to improve the properties of concrete; however, they should never be used as a substitute for proper design of the concrete mix or proper construction procedures. Also, some admixtures may not be compatible with the other materials used in the concrete. Loss of workability, loss of strength, and excessive shrinkage causing cracking may result. Therefore, it is important that concrete trial mixes are made with the admixture and at temperatures anticipated on the job. In this way, the effects of the admixture can be observed. The amount of admixture is recommended by the manufacturer and verified through tests. Following is a discussion of the concrete admixtures and the reasons for using each.

5.2.4.1 Air-Entraining Admixtures

Recommendation

An air-entrained admixture is recommended for use in all concrete used in structures that are exposed to freezing temperatures

An air-entrained admixture is recommended for use in all concrete used in structures that are exposed to freezing temperatures because it improves the hardened concrete's resistance to surface scaling caused by freezing/thawing action and deicing salts. When the water in the concrete surface freezes, it expands and ruptures the surface layer causing scaling. The entrained air voids act as reservoirs for excess water forced into them. As a result the pressure on the concrete is relieved and damage is prevented. However, air-entrained admixtures have other beneficial effects in both plastic and hardened concrete.

These beneficial effects are:

- Workability of concrete is improved, especially in lean mixes (those with low cement content) because air bubbles act as rollers. Air entrained concrete is cohesive, concrete flows sluggishly, and its segregation is reduced.
- Impermeability of concrete is improved. This is because air entrained concrete uses lower amounts of water in the mix for the same workability. Thus, the voids caused by the evaporation of the excess water, which is not used for hydration, will be minimal.
- Resistance of concrete to erosion caused by sulfate attack is improved. Air-entraining agents protect against sulfate attack in the same manner as they protect against freezing and thawing. The air voids provide microscopic expansion chambers for relief of the pressure that is built up in the concrete by the growth of the sulfate crystals. Air entrainment does not provide permanent protection, but it does delay the deterioration for a period of time depending upon factors such as the concentration of soluble sulfate and the cycles of wetting and drying, which promote the crystal growth.

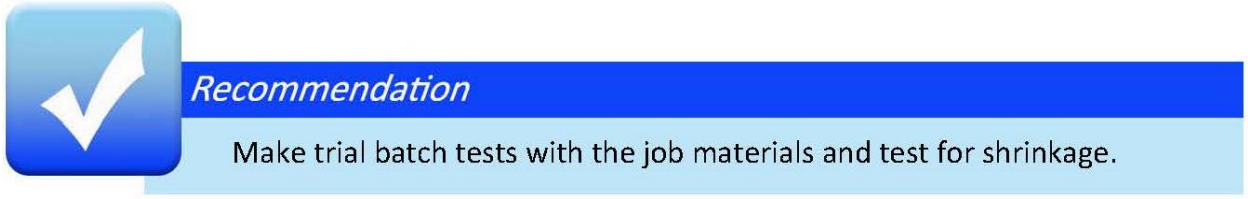
An air-entrainment admixture is introduced into the batch of concrete with water before mixing the concrete. Air-entrained concrete contains microscopic air bubbles that are distributed, but not interconnected, through the cement paste. Hence, these voids do not act as capillaries and do not affect the permeability adversely. The bubbles are small and invisible to the naked eye. The entrained air is typically from 5 to 8 percent of the volume of concrete. Generally as the maximum aggregate size increases, the amount of entrained air required decreases.

5.2.4.2 Water-Reducing Admixtures

Addition of a water-reducing admixture to concrete mix will reduce the quantity of mixing water required to produce concrete of a given workability. An increase in strength can be obtained with a water-reducing admixture when the water content is reduced and the cement content is kept the same.

Water-reducing admixtures increase the workability of concrete if the same quantity of water is used in the mix. Many water-reducing admixtures also retard the setting time of concrete. However, the time duration of increased workability is short.

A significant increase in drying shrinkage of concrete can result from the use of some of these admixtures. Drying shrinkage takes place after the concrete has gained strength and the excess water has left the voids. The shrinkage in turn can cause cracking of the concrete. It is, therefore, recommended that trial batch tests be made with the job materials and tested for shrinkage.



5.2.4.3 High-Range-Water-Reducing-Admixtures

These admixtures are commonly known as super-plasticizers. Basically, they perform similar to water-reducing admixtures. The difference is that their addition to the mix will allow a significant reduction in the quantity of the mixing water required to produce concrete of a given workability. This will permit producing concretes with extremely low water-cement ratio (water-cement ratio lower than 0.4) and very high strengths. Superplasticizers may have adverse effects on concrete drying shrinkage; therefore shrinkage tests on trial mixes should be conducted.

5.2.4.4 Retarding Admixtures

A retarding admixture is a chemical that is used to delay the setting time of concrete. This allows time for the finishing of concrete when, as in hot weather, the set time is reduced. It also provides time to place concrete in difficult places or allows time for finishing of such items as bridge decks or large piers. Most retarders also function as water reducers. The effects of retarding admixtures on concrete drying shrinkage may not be predicted, and shrinkage tests of retarders should be conducted.

5.2.4.5 Accelerating Admixtures

An accelerating admixture is used to decrease the set time and allow for early strength development of the concrete. There are benefits of accelerated setting and early strength gain. In cold weather, cement hydrates slowly, and concrete sets and gains strength slowly. Concrete protection is necessary during the curing to avoid damage by freezing. This protection can be in the form of heating the concrete. Accelerating admixtures can significantly reduce the duration of this costly protection against cold weather. Also, in cold weather, forms can be removed

earlier by using accelerating admixtures. Accelerating admixtures are also useful in bridge deck repair, where is desirable to reopen repaired areas to traffic as soon as possible.

In the past, calcium chloride was one of the most commonly used accelerating admixtures. It has been determined, however, that chlorides can cause corrosion of the reinforcing steel and spalling of concrete. Thus, most highway agencies, if not all, have discontinued its use. With the elimination of calcium chloride as an accelerating admixture, the use of Type III cement (high-early-strength cement) has become more common. Another alternative is the use of additional cement in the mix, or the use of different methods of curing and protection such as heating concrete. A substitute for heating concrete may be heating the water and aggregates prior-to-mixing. However, in many cases the use of an accelerator is the most economical and convenient method of obtaining the desired results.

5.2.4.6 Mineral (Cementitious) Admixtures

Cementitious materials, such as fly ash, slag, and microsilica are used both as an admixture and as a partial substitute for cement in concrete.

These admixtures are used to:

- Reduce permeability
- Reduce heat of hydration
- Reduce or eliminate potential expansion from alkali-reactive aggregates and the subsequent cracking of concrete
- Add to the strength
- Most importantly, they are cheaper than cement

The advantages to using mineral admixtures added at the batch plant:

- Mineral admixture replacement levels can be modified on a day-to-day and job-to-job basis to suit project specifications and needs.
- Cost can be decreased substantially, while performance is increased, when taking into consideration that the price of blended cement (defined as a mixture of Portland cement and other pozzolans such as Ground Granulated Blast-Furnace Slag (GGBFS), hydrated lime, etc., combined either during or after the finish grinding of the cement at the mill) is at least 10 percent lower than that of Type I/II cement.
- Ground Granulated Blast Furnace Slag (GGBFS) can be ground to its optimum fineness.
- Concrete producers can provide specialty concretes in the concrete product markets.

At the same time, precautions must be considered when mineral admixtures are added at the batch plant. One example includes the possibilities of cross-contamination or batching errors are increased as the number of materials that must be stocked and controlled is increased.

Fly Ash

Fly ash is a pozzolanic material, which means that when in small particle form and under the right conditions, it reacts with calcium hydroxide and water at ordinary temperatures to form a cementitious material. Fly ash may be added either as a replacement for a portion of the Portland cement or as an addition to the usual amount of cement. Fly ash content is generally between 15 to 20 percent of the weight of cement. Fly ash improves the workability of concrete. The permeability of concrete with fly ash generally decreases with time and its

strength increases because of the pozzolanic reactions that occur during and also after the curing period. The pozzolanic reactions depend on the temperature. Those reactions are slow in cold temperatures. If fly ash is replacing the cement in concrete, in cold temperatures, curing periods longer than those required for conventional concrete may be needed to obtain the required strength.

Slag

Ground slag may be used as a replacement for a portion of cement in concrete. The slag content is generally between 40 to 50 percent of the cement weight. A properly used good quality slag reduces concrete permeability in time and adds to the strength. Ground slag also improves the workability of concrete.

Microsilica

Microsilica (silica fume) is a highly effective pozzolanic material. It is obtained as a by-product of the manufacture of silicon. Microsilica admixtures for concrete are furnished either in the dry compacted form or as water-based slurry that are made from dry material. The water in the slurry shall be counted as mixing water for the purpose of determining the water-cement ratio of the concrete.

Addition of microsilica to concrete reduces the permeability significantly. In many cases, the permeability is so low that existing equipment is not suitable for measuring permeability of microsilica concrete. Addition of microsilica substantially increases the strength; however, achieving low permeability is the primary reason for its use in bridge components. Adequate impermeability and resistance to chloride penetration for concrete used in highway applications may be attained with microsilica dosages between 5 to 10 percent of cement weight.

Adding microsilica to concrete reduces slump and adversely affects workability. This is because extremely fine microsilica particles increase water demand. Thus, the use of microsilica in concrete mix almost always makes it mandatory to use HRWRs. The HRWR will compensate for the increased water demand and will allow placement of the concrete at reasonable field slump. Some states have had issues with microsilica concrete overlays bonding to the underlying concrete, so microsilica overlays should be carefully designed and placed for successful performance.

Concrete containing microsilica are highly susceptible to plastic shrinkage cracks during the curing process. Special concrete curing procedures must be set up to ensure proper curing of concrete.

5.2.4.7 Corrosion Inhibiting Admixtures

Recommendation

Perform strength testing on concrete containing inhibitor to ensure that the inhibitor does not adversely affect the concrete strength.

Corrosion inhibiting admixtures provide protection for steel embedded in concrete by reducing the corrosion rate in the presence of chloride ions. Inhibitors act at the steel surface to limit corrosion reactions. Traditionally, inhibitors were inorganic. More recently, inorganic inhibitors have become available. To ensure that the inhibitor does not affect the basic properties of the concrete, concrete containing the inhibitor should be tested for strength.

Inorganic Corrosion Inhibitor

The active ingredient in inorganic corrosion inhibitors is calcium nitrite. The calcium nitrite inhibitor can be added at varying amounts to provide different levels of protection. Calcium nitrite is typically added at a rate of 3 to 5 gallons per cubic yard depending on the severity of deicer salt application. The water in the calcium nitrite should be counted as mixing water for the purpose of determining the water-cement ratio of the concrete.

Calcium nitrite may act as an accelerator, depending on type used, and adjustment in quantities of other admixtures to produce acceptable workability and setting time may be required. The calcium nitrite solution shall be added immediately after the other admixtures have been incorporated into the mixture. However, if High-Range Water Reducer (HRWR) is added at the site, the calcium nitrite shall be thoroughly mixed before the HRWR is added.

Organic Corrosion Inhibitor

Organic corrosion inhibitors are relatively new products. The admixture functions by a twofold mechanism that involves the formation of a protective film at the steel surface against corrosion and a reduction in the permeability of concrete to chloride penetration. The organic corrosion inhibitor will not accelerate the rate of setting of concrete and has no effect on slump, thereby allowing normal placing and finishing operations.

5.3 Polymer Concrete

Polymer concrete as a repair material for highway infrastructures is relatively new. While traditional concrete uses cement as a paste to bind the aggregates together, polymer concrete uses non-Portland Cement based chemical compounds, called polymers, to bind the aggregates together.

Polymer concrete offers several distinct advantages over traditional concrete. It is quick curing, versatile, and is able to expand and contract over time to accommodate temperature changes. The adhesive properties of polymer concrete allow patching of both polymer and conventional cement-based concretes. The low permeability and corrosive resistance of polymer concrete allows it to be used in bridge repairs, culverts, and roadways.

Polymer concrete has historically not been widely adopted due to the high costs and difficulty associated with traditional manufacturing techniques. Uses have been limited to small

placements or when reopening the structure as soon as possible is critical. However, recent progress has led to significant reductions in cost, meaning that the use of polymer concrete is gradually becoming more widespread. While these historically high costs have limited the overall thickness of the polymer repair, there is no set limit on the thickness. However, as the compressive strength is typically less than that of Portland cement concrete, deep patches of polymer may be more susceptible to rutting and debonding due to differences in thermal expansion. Even though polymer concretes can be formulated to 12,000 psi, typically lower strength polymer concretes are used (1,500 to 2,500 psi) to facilitate field mixing and application of polymer concretes for maintenance repairs and patching.

Polymer concretes used in bridge repair are “elastomeric polymers”, meaning the material has elastic properties. As defined by ASTM, an elastomer is "a polymeric material which at room temperature can be stretched to at least twice its original length and upon immediate release of the stress will return quickly to approximately its original length".

Elastomers are rubbery material composed of long chainlike molecules, or polymers, that are capable of recovering their original shape after being stretched to great extents. Under normal conditions the long molecules making up an elastomeric polymer are irregularly coiled. With the application of force, however, the molecules straighten out in the direction in which they are being pulled. Upon release, the molecules spontaneously return to their normal compact, random shape. Elastomers are hydrocarbon polymeric materials similar in structure to plastic resins. The difference between plastics and elastomers is based on the property of extensibility or stretching. Elastomers have long been praised for being very strong when struck, hard if scratched, resistant to corrosion from various chemicals, and resilient in the face of humidity or water submersion. Since they do not conduct current, they are good electronic insulators.

Another beneficial property is its ability to be compounded, or mixed, with other materials to strengthen certain characteristics. Other kinds of polymers may make them less likely to soften at high temperatures or break down around ozone gas. Elastomers can easily be installed next to various other materials, such as metal and concrete, with excellent adherence.

5.3.1 Advantages and Disadvantages of Polymer Concrete

5.3.1.1 Advantages of Polymer Concrete

There are many advantages of polymer concrete, including:

- Rapid cure times. Polymer concrete has a liquid working temperature between 60 °F and 85 °F. The maximum workable temperature of this type of concrete is 150 °F. However, at higher temperatures, these materials can set in minutes and be hardened in the mixer.
- High tensile and flexural strengths. These materials can bend and twist more than traditional concretes.
- Good adhesion to most surfaces. Polymers are sticky and will develop strong bonds to properly prepared surfaces, such as concrete and steel.
- Good long term durability with respect to freeze-thaw cycles. The material is very dense so it does not allow water into the pours as it can with Portland-based materials. Polymer concrete does not need to be sealed.
- Low permeability to water and aggressive solutions such as de-icing salts.

- Good resistance against corrosion.

5.3.1.2 Disadvantages of Polymer Concrete

Polymer concrete must be mixed very precisely and very thoroughly. It cannot be mixed beforehand and simply kept turning to avoid curing because the chemical reaction will happen no matter what. Also, the chemicals that this type of concrete uses can be dangerous and everyone nearby must wear masks and skin protection.

Additional disadvantages of polymer concrete include:

- Cost. Polymer concrete is expensive. Too expensive for use in large placements.
- Lower compressive strength. Though polymers can be formulated to 12,000 pounds of pressure per square inch, or psi, the elastomeric concretes used in maintenance repairs are typically of lower strengths, typically 1,500 to 2,500 psi.
- Polymer concretes are sticky and hard to manipulate with conventional tools shovels and trowels.
- Not compatible with water. Most polymers are negatively affected by water. Both the substrate and the aggregate should be free of any water. Some polymers are affected by high humidity.
- Safety. Some polymers are relatively safe to use, others can create hazardous conditions if mixed improperly.
- Training. Crews should receive specific training and review the Material Safety Data Sheet (MSDS) before mixing and placing polymer materials.
- Temperature limitations. Most, but not all polymers, cannot be used in below freezing temperatures. Also, at high temperatures, the material may set too quickly.
- Surface Preparation. Proper surface preparation is critical for quality repair.

5.3.2 Components of Polymer Concrete

5.3.2.1 Polymers

Polymers are typically made up of two chemical compounds. One is termed a resin and the other a hardener. When the two components are mixed together, a chemical reaction occurs and the material begins to cure and harden, thereby, becoming the binder and binding the aggregates together. Some polymers include a third component called an initiator (or accelerator) that helps the polymerization reaction to begin.

Manufacturers have also developed single component systems called monomers that are added to cement to create polymer-impregnated concretes and polymer-Portland cement concrete.

Many different types of polymers can be used to make up polymer concrete, each with specific properties and characteristics. Acrylic, polyesters, polyureas, and epoxies are most often used, but polymer concrete can be made with many kinds of polymer resins that allow the repair material to be pumped, poured, or troweled and then hardened. Polymer concrete offers different benefits depending on the resin used to make it.

Acrylic binders set very quickly and offer resistance to weathering, while epoxies create a very strong material that shrinks very little as it cures. Furan resins can withstand high temperatures, and polyurea resins can replace phenolics or formaldehydes in many construction projects.

Single component monomers, such as latex, have been used in place of water to make latex modified concrete.

5.3.2.2 Aggregates

Polymer concrete is composed of aggregates that include silica, quartz, granite, limestone, and other high quality material. The aggregate must be of good quality, free of dust and other debris, and dry. Failure to fulfill these criteria, especially for a dry aggregate, will reduce the bond strength between the polymer binder and the aggregate.

5.3.3 Material Properties of Polymer Concrete

5.3.3.1 Compression Strength

Strength is an important characteristic of concrete, as it is often used as a structural foundation material. The compressive strength of elastomeric polymer concrete can range from 1,500 to 12,000 psi. Exact compression strength varies depending upon the brand of polymer concrete and what type of material is used as a binder. Compressive strength increases over time. Polymer Methyl Methacrylate Concrete can gain compressive strengths of 8,000 psi after one hour of setting and 12,000 psi after seven days of setting.

5.3.3.2 Corrosion Resistance

A main advantage of polymer concrete is resistance to a wide range of solvents and oils; it is also resistant to acids (pH level ranging from 0 to 7). Polymer concrete is resistant to a variety of acids that can eat away concrete over time, de-icing salts, hydrochloric acid, formic acid, and ammonium sulfate. Polymer concrete does not corrode in the presence of hydrocarbons, such as acetone, gasoline, ethane, methanol, petroleum, diesel and vegetable oils. This type of concrete offers excellent resistance against water.

5.3.3.3 Flexibility

Traditional concrete is vulnerable to shearing forces that can cause cracks and a decline in structural integrity. Resistance to these forces is known as flexural and tensile splitting strengths.

Recommendation

When using polymer concrete, it is important to follow the manufacturer's recommendations. Product representatives may be available to provide training and assist with the proper use.

Polymer concrete offers greater resistance to cracking. Polymer concrete has a flexural strength of around 3,500 psi after eight hours compared to less than 1,000 psi for normal concrete. Tensile strength is around 2,000 psi after the material has been allowed to set for seven days, whereas normal concrete typically has just a few hundred psi. Alternatively, for patching, there are numerous proprietary brands of quick-setting cement. These can be used for patching, but

a normal requirement is that they be placed in relatively thin layers because of the heat they generate. The manufacturer instructions should be followed.

5.3.3.4 Using Polymer Concrete

Choose a liquid or dry redispersible polymer option, depending on your needs and preferences. Liquid polymers usually come in a jug with instructions for how much to add to a bag of dry mix containing sand, cement, and other ingredients. It will also include instructions for avoiding freezing temperatures; this affects the integrity of the polymer. Dry polymers are usually added to the dry mortar mix in advance, so the contractor only needs to add water to a bag of dry mix.

Follow the instructions on the label: Cutting corners, like using too little polymer from the jug or using a product in an application for which it was not designed will likely lead to a failed repair.

The use of polymer concrete in concrete replacement is covered in Section 5.7.6.

5.4 Other Types of Concrete

The two previous sections focused on the most popular types of concrete used in bridge maintenance (hydraulic cement concrete and polymer concrete). Other types include:

- High Performance Concrete
- Self-Consolidating Concrete
- Lightweight Concrete

High performance concrete is a specialized series of concretes designed for better performance, cost, extended life cycle and most importantly reduced maintenance. Performance benefits applicable to bridge maintenance include early high strength, toughness, ease of placement and longer life in severe environments. Examples types include Very Early Strength Concrete, which can achieve a compressive strength of 2,000 psi in only 6 hours, to Very High Strength Concrete, which can achieve a compressive strength of 10,000 psi in 28 days. Some mixes include special admixtures such as fiber reinforcement to enhance strength and durability.

Self-Consolidating concrete is highly flowable, resistant to segregation, and requires no vibration. This is an excellent choice for bridge maintenance repairs where there is no effective way to vibrate the concrete. It is also an excellent choice where there is significant rebar congestion that might lead to voids and honeycombing of the repair. Self-Consolidating concrete uses new generation admixtures and for that reason is more expensive than normal concrete. It has some drawbacks, such as sensitivity to water ratio changes in the mix. An example of a slump test, used to determine the workability of fresh concrete, is shown in Figure 5.1. The slump test for self-consolidating concrete is very different from normal concrete. Because the mix is so flowable, the spread of the mix is measured instead of the slump height. The spread of the mix is measured by measuring the diameter of the poured concrete cone

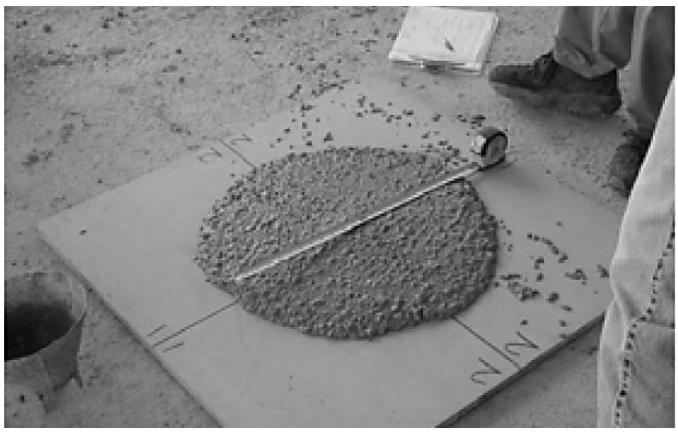


Figure 5.1 Self Consolidating Concrete Slump Flow Test

Lightweight concrete uses a lighter weight aggregate and can reduce overall concrete weight by 20 percent. It could be considered for a concrete repair where the weight of the normal concrete repair could be damaging to the bridge element. Lightweight concrete has been used in many decks. Due to the different types of aggregates used in lightweight concretes and the need for compatibility of the repair with the original design/materials (e.g., weight, electric potential, etc.), an engineer should be consulted if a repair to a lightweight concrete bridge member is to be made.

When to Call the Engineer

Call the Engineer when a repair to a lightweight concrete bridge member is going to be made.

5.5 Identification of Deteriorated Concrete

Concrete is a cost effective building material, i.e., relatively cheap but strong (in compression) and fairly chemically resistant. But eventually concrete will crack and deteriorate. There are many reasons why deterioration occurs, such as:

- Adverse chemical reactions between the cement and the aggregate
- Improper mix design
- Lack of concrete cover
- Problems with delivery to the site
- Temperature extremes on the day of the pour
- Poor curing

While these issues are all important, even a quality mix design placed in perfect conditions can still have problems down the road.

The culprit in concrete degradation typically is not the concrete itself, but the steel reinforcement (rebar) that is embedded in the concrete. When water has access to the rebar, either through a crack in the concrete or through pores in the concrete itself, the rebar is then exposed to water, oxygen, and potentially chlorides. These ingredients, in physical contact with the steel rebar, will begin the corrosion process. Once steel starts to corrode, it expands to

many times its original cross section due to the formation of corrosion product at the rebar surface. The corrosion product exerts pressure onto the concrete, a tension force, and since concrete is weak in tension, it will further crack. These new cracks will allow more water, oxygen and salt to further corrode the rebar in a vicious cycle. Soon the concrete will delaminate or debond from the steel, and a spall develops. See Figure 5.2 that depicts the typical concrete deterioration process.

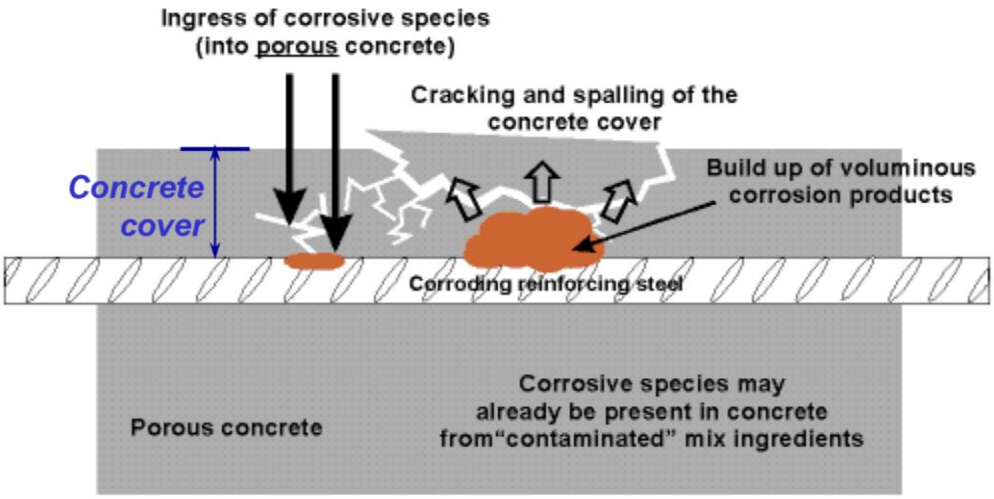


Figure 5.2 Concrete Deterioration Process

Engineers have tried many kinds of potential solutions to this problem. A summary of potential strategies for preventing concrete deterioration is presented in Table 5.2.

Table 5.2 Potential Solutions to Prevent Concrete Deterioration

Potential Solution	How is the Solution Working?
Eliminate one of the ingredients that cause rebar corrosion. Do not salt the concrete roads and bridges.	Salt (chloride) is excellent at melting ice and snow. Some states use sand, but when the sand pile freezes, they add salt to melt. Don't introduce chloride-containing salts to concrete.
Protect the steel rebar by coating	This has been used with some success, epoxy, galvanized, stainless steel, stainless clad rebars, etc. Caution needed when rebar coating is damaged, particularly during on site storage and placement during bridge construction (e.g., there are stories of trucks driving over the rebar).
Replace steel rebar altogether with smart materials	Rebar made of fiber reinforced composites (FRP) has been used in many bridges with promising results.
Make concrete more dense	Low permeability concrete has helped deter the corrosion process by reducing the water access to the rebar.
Preventive Maintenance, fill cracks as soon as they appear, apply protective membranes	BINGO!

Failure of concrete is a complex phenomenon because more than one mechanism of damage occurs at the same time, and it is difficult to determine which problem caused the initial damage. It is important to have an understanding of the basic underlying causes of damage in concrete and the maintenance problems that develop. The most common cause for failure of reinforced concrete is corrosion of the reinforcing steel, but concrete can deteriorate by other means as well.

Some common construction related defects in concrete include voids and inclusions (also known as honeycombing, due to incomplete vibration), poor surface finish or cracked surface (due to plastic shrinkage), damage and cracking from residual stresses (due to thermal effects), surface weakness, weak bonds between steel and concrete due to bleeding and cracking at cold joints.

In-place concrete can deteriorate from load-related causes, such as fatigue, impact, residual stresses due to overloading, and creep. Or, deterioration can be environment-related, such as corrosion, chemical attack (alkali-silica reaction), creep and shrinkage, carbonation, freezing and thawing, de-icing salts, and scaling. Improper maintenance or repair could also be classified as a service-induced damage.

5.5.1 Condition Assessment

Assessing the condition of concrete members is categorized into Non-Destructive Testing (NDT), semi-destructive testing, and destructive testing. NDT (also called Non-Destructive Evaluation or NDE) is a class of evaluation techniques that do not damage the existing concrete. Semi-destructive tests require removal of a piece of the material for evaluation. Destructive tests are performed to failure. Destructive tests, such as those done on concrete cores, result in some minor damage to the concrete, but the damage can be repaired. A brief summary of NDE is provided in this section, with a more comprehensive summary provided in Appendix 2 References.

The first step in performing a condition assessment is to get details of the structure with respect to its design, features, and past performance. An initial visual inspection of the structure can reveal useful information about areas that need a closer look. There are many causes for the deterioration of structures, so it is difficult to exactly pinpoint the type of damage that has led to the deterioration. However, all types of damage, whether they be load related, environment related, or earthquake related, lead to similar signs of concrete deterioration, cracking, spalls, delaminations, and corrosion.

Traditional methods for locating unsound or deteriorated concrete include visual assessments (including cameras), sounding (chain drag and hammer sounding), and half-cell potential measurements (copper-copper-sulfate). Specialized equipment can also be brought in to help determine the specific mode of deterioration. These methods are discussed further in Section 5.5.3, Chapter 20, and Appendix B.

5.5.2 Types of Reinforced Concrete Deterioration and Detection

The main categories of reinforced concrete deterioration are shown below:

Cracking

Cracking can be found in almost all concrete bridge elements. The cracks may be vertically oriented such as the column cracks shown in Figure 5.3, horizontally oriented following the rebar spacing, and they can be full depth or partial depth such as deck cracks due to concrete plastic shrinkage or restraint. Cracking can also be random based on the material deterioration as described below.

Delamination

Detection of delaminations (or the debonding of concrete from the reinforcing steel) includes detection of predominantly horizontally-oriented cracks, although substructure members may exhibit delamination on vertical surfaces. Delamination occurs as a result of rebar corrosion, overloading, or other type of deterioration. Delaminated concrete needs to be detected and restored to original conditions. Delaminations are detected by sounding.

Spalls

Spalls (concrete pieces that are flaking or breaking away from the surface) are detected during a visual review.

Corrosion

Detection of rebar corrosion is directed towards two main objectives: detection and evaluation of intensity of active corrosion, and detection and evaluation of the severity of existing corrosion.

Figure 5.3 Vertical Crack on a Pier Column

Concrete Material Deterioration

Detection of concrete deterioration involves measurement of a change or variability of material properties regardless of what is the cause of concrete deterioration: corrosion, alkali-silica reaction (ASR), carbonation, and mechanical or thermal stresses.

Alkali Silica Reactivity

ASR is a widespread problem, affecting concrete structures in almost every area of the U.S. It occurs when silica in aggregate and alkali in cement react in the presence of water. This reaction creates a gel in the hardened concrete. Over time this gel expands causing cracks and disintegration of the bond between the concrete ingredients. An example of ASR cracking is shown in Figure 5.4.

The *SHRP Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures*, http://www.fhwa.dot.gov/pavement/concrete/asr/pubs/hif12022.pdf, describes a test that treats concrete core samples with a uranium acetate solution before viewing under ultra violet (UV) light. Any ASR gel present in the treated concrete will then appear to fluoresce under UV light. An example of ASR cracking viewed by this method is shown in Figure 5.5.

Figure 5.4 ASR Cracks in a Concrete Column

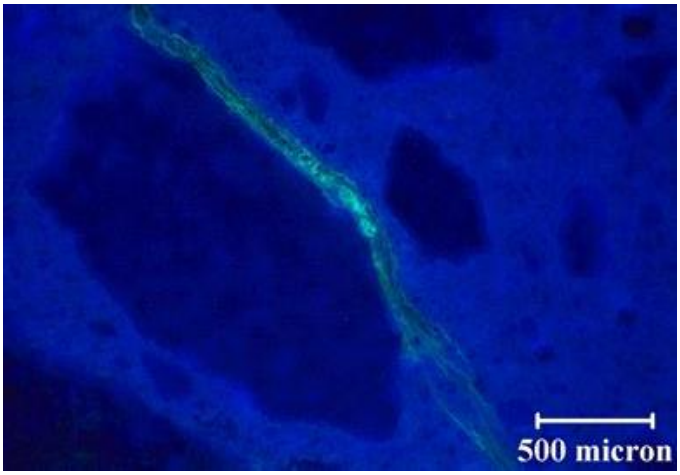


Figure 5.5 Microscopic Image of ASR Crack

Reactive aggregates should be avoided if possible in future construction and repairs. However, solutions to prevent or mitigate this problem on existing bridges are limited. The most practical solution appears to be drying concrete (artificially if possible) and sealing with a low viscosity polymeric material (such as high-density methacrylate) which could penetrate deep into hairline cracks. The application of this method is limited to the elevated members, such as bridge decks. In that case, sand is spread on the sealer for friction. In members in contact with earth the drying method may not be effective because of the ready access of moisture from the earth. For bridge columns, it has been suggested that ASR expansion may be restrained by cylindrical wrapping of the column.

For each of the main deterioration types, several technologies have shown potential for detection and evaluation. However, there is not a single technology that has shown potential in evaluation of all deterioration types.

For additional information, please refer to Chapter 20 Concrete Deck Decision Aid Matrices and Appendix B References.

5.5.3 Non-Destructive Evaluation (NDE) Techniques

Non-destructive evaluation techniques range in sophistication from simple methods where the quality of sound obtained by striking the surface of the material with a hammer to complicated techniques where the ultrasonic signals traveling through the material are analyzed mathematically.

Good NDE methods possess the following qualities:

- Sensitivity to small flaws
- Reliability
- Simplicity
- Cheap
- Portable
- Easy to use

NDE methods provide information but are not 100 percent accurate. Some factors that affect the reliability of NDE techniques are:

- The location and the orientation of subsurface cracks
- Difficult access for the equipment
- Selection of the proper technique
- Correct application – proper training of technician and proper calibration of equipment
- Environmental factors – weather, live traffic, and material property
- Human factors – the location of the test, operator fatigue, time constraints, bias, and experience

5.5.3.1 Visual Assessment

Visual assessment is the most commonly used non-destructive evaluation method for determining concrete condition. A detailed visual inspection makes it possible to focus on the critical areas in a structure that may benefit from further investigation using more sophisticated techniques. Areas that show cracking, discoloration, rust stains, etc. should be investigated closely with the help of visual aids such as magnifying lens and crack comparators. Visual inspections can identify areas of deterioration that could be investigated further using more sophisticated techniques. Concrete scaling, such as that shown in Figure 5.6, is one of the forms of deterioration readily identifiable during a visual inspection.

Visual inspections can be aided by a number of tools such as:

- Telescopes / Binoculars
- Camera
- Magnifying lenses
- Real-time video
- Ruler, measuring tape
- Crack width gauge comparator
- Light hammer, chipping / scraping tools

Figure 5.6 Scaling on a Bridge Deck (Courtesy of Iowa State University)

Table 5.3 provides a visual observation of concrete deterioration and corresponding probable cause.

Table 5.3 Likely Causes of Various Types of Deterioration

Visual Observation	Deterioration Type and Probable Cause
Rust staining, cracks run in straight parallel lines at uniform intervals as per the reinforcement position. Spalling of concrete cover	Reinforcement corrosion: Exposure to normal atmospheric conditions, Cyclic wetting and drying
Cracks mostly on horizontal surfaces, parallel to each other, 3 to 6 feet apart, relatively shallow 1/2 to 1 inch in depth and vary in length from 1 inch to 10 feet (see Figure 5.7)	Plastic shrinkage: Caused by surface tension forces, environmental effects of temperature (concrete and ambient), wind velocity in excess of 5 mph and low relative humidity while curing
Cracks characterized by their fineness and absence of any indication of movement, shallow (a few inches) in depth, typically orthogonal or blocky	Drying shrinkage and creep: Placement of a footing on a rough foundation, or chemical bonding of new concrete to earlier placements; the combination of shrinkage and restraints cause tensile stresses that can ultimately lead to cracking
Cracks are regularly spaced (restrained contraction) and perpendicular to larger dimensions of concrete, spalling (restrained expansion), shallow and isolated (internal restraint), extend to full depth (external restraint), surface discoloration (fire damage) (see Figure 5.8)	Thermal effects: Induced by exothermal chemical reaction in mass concretes. If volume change is restrained during cooling of the mass, by the foundation, the previously placed concrete, or exterior surfaces, sufficient tensile strain can develop to cause cracking
Spalling and scaling of the surface, exposing of aggregate which is un-cracked, surface parallel cracking and gaps around aggregate (see Figure 5.6)	Freeze-thaw deterioration: Alternate cycles of freezing and thawing, use of deicing chemicals
Absence of calcium hydroxide in cement paste and surface dissolution of cement paste exposing aggregates	Acid attack: Acid smoke, rain, exhaust gases
Rough surface, presence of sand grains (resembles a coarse sand paper)	Aggressive water attack: Causes serious effects in hydraulic structures due to a continuous water supply resulting in the washing away of aggregate particles due to leaching of cement paste
Map or pattern cracking, general appearance of swelling of concrete (see Figure 5.9)	Alkali-carbonate reaction: Chemical reactions between alkali in cement with certain dolomitic aggregates, Expansion due to dedolomitisation and subsequent crystallization of brucite
Map or pattern cracking, expands freely, silica gel leaches from cracks, calcium hydroxide depleted paste.	Alkali-silica reaction: Chemical reactions between alkali ions (Na+ and K+) in cement with silica in aggregates
Map and pattern cracking, general disintegration of concrete	Sulphate attack: Formation of gypsum, thaumasite and ettringite which have higher volumes than the reactants
Single or multiple long diagonal cracks (usually larger than 0.25 inch in width) accompanying misalignment and displacements	Structural damage: Induced by improper construction and maintenance throughout the lifetime of a structure.
Spalling or cracking of concrete, complete collapse of structure	Accidental loadings: Generates stresses higher than strength of concrete resulting in localized or complete failure of the structure
Honey combing, bug holes (Small holes less than about 0.25 inch in diameter), cold joints, cracking in concrete	Construction errors: Improper mix design, consolidation, curing etc., inexperienced labor work, incorrect position of reinforcement.
Surface is generally smooth with localized depressions, long shallow grooves, spalling along monolith joints (abrasion). Severely pitted and extremely rough surface (Cavitation)	Erosion: Rolling and grinding of debris (abrasion), sub atmospheric pressure, turbulent flow and impact energy (cavitation)
Cracking or spalling of concrete, complete deterioration of the structure. (see Figure 5.10)	Design errors: Abrupt changes in design, insufficient reinforcement, inadequate provision for deflection and drainage

The following forms of deterioration may be observed by visual inspection and apply to concrete construction:

- Crack type, classified as:
  - Hairline – barely visible
  - Fine – 1/32 inch to 1/16 inch
  - Medium – 1/16 inch to 1/8 inch
  - Wide – Greater than 1/8 inch
- Patterns, location, and orientation of cracks (whether stress related or not)
- Scaling, classified as:
  - Light – does not expose coarse aggregate
  - Medium – involves loss of surface mortar to 1/8 inch to 3/8 inch in depth and exposure of coarse aggregate
  - Severe – involves loss of surface mortar to 1/8 inch to 3/8 inch in depth with some loss of mortar surrounding aggregate particles 3/8 inch to 5/8 inch in depth
  - Very Severe – involves loss of coarse aggregate particles as well as mortar generally to a depth greater than 5/8 inch
- Spalling, classified as:
  - Small – involves a roughly circular depression not greater than 5/8 inch in depth and 6 inch in any dimension
  - Large – may be roughly circular or oval or in some cases elongate; more than 5/8 inch in depth and 6 inches in greatest dimension
- Exposed reinforcement

Other general defects that can be identified by visual observation are:

- Surface distress: Disintegration of the surface, surface honeycombing.
- Water leakage: Surface dampness, efflorescence, seepage or leakage through joints or cracks.
- Movements: Deflection, heaving, settlement.
- Metal corrosion: Rust staining, exposed post-tension cable strands.
- Miscellaneous: Blistering membranes and coatings, ponding of water, discoloration.

Figure 5.7 Plastic Shrinkage

Figure 5.8 Cracking from Thermal Effects

Figure 5.9 Map Cracking

Figure 5.10 Spalls and Cracking

Concrete Cracks

Cracks in reinforced concrete do not necessarily mean the presence of unsound concrete. Regardless of the crack type, water seeps in and causes the reinforcement to corrode. The corroded reinforcement expands and exerts pressure on the concrete. This pressure can cause delaminations and spalls. The presence of rust stains or efflorescence are evidence of possible reinforcement corrosion and section loss.

Exercise care in distinguishing between nonstructural cracks and structural cracks. An example of spalled concrete and efflorescence is shown in Figure 5.11.

Figure 5.11 Spalled Concrete and Efflorescence on the Underside of a Bridge Deck

Crack widths in reinforced concrete bridges exceeding 0.006 inches to 0.012 inches reflect the lower limit of moderate cracking. The American Concrete Institute Committee Report 224R-01 presents guidance for what could be considered reasonable or tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions. These range from 0.006 inches for marine or seawater spray environments to 0.007 inches for structures exposed to de-icing chemicals, to 0.012 inches for structures in a humid, moist environment. A crack comparator used for determining crack widths is shown in Figure 5.12.

In prestressed concrete bridge structural elements, cracking is generally more problematic than cracking in reinforced concrete elements. This is because prestressing is designed to control or prevent cracking in the member while normal steel reinforcement is intended to structurally bridge cracks. Thus, cracking in prestressed elements may indicate a design or fabrication flaw or that an overload of the structural element has occurred. A determination of the exact cause of the cracking, as well as an assessment of possible structural damage to the element and/or its prestressing steel, is needed before arriving at a repair method.

Figure 5.12 Crack Comparator for Determining Crack Width

5.5.3.2 Sounding

Hammer Sounding

A good evaluation of concrete can be easily obtained by just sounding it (i.e., tapping it) with a hammer. When the hammer is struck on good concrete, a ringing sound is created. Where delaminations occur, the striking of the hammer produces a drum-like, or thud. A schematic is presented in the left side of Figure 5.13. Because sounding is a manual method, several factors can affect accuracy of the investigation. Exposure to constant sounding may lead to temporary or permanent operator desensitization to high-pitched tones. It can also be difficult to hear when sounding near live traffic. The feel of the rebound of the hammer will help an experienced technician determine the condition of the concrete. The hammer will seem to bounce off sound concrete but tend to "lay-into" punky concrete. The limitation of sounding is that it cannot detect defects that exist deep in the member. Also, defects lying under overlays are also difficult to find. Operator fatigue can also be a problem.

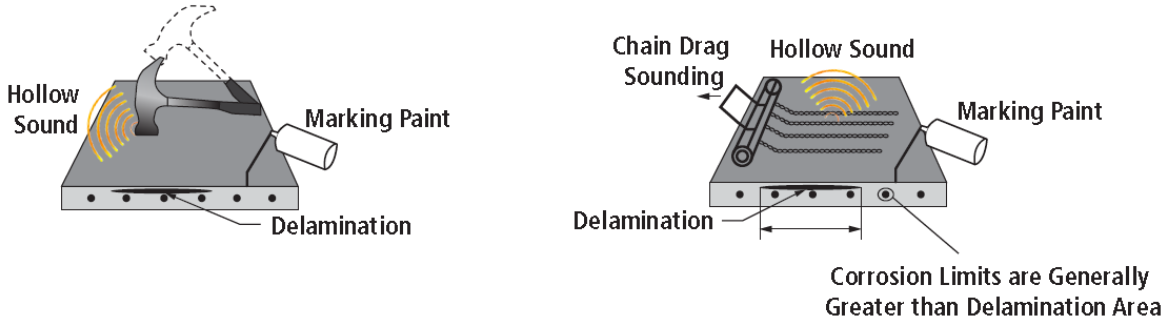


Figure 5.13 Sounding Tests: Hammer (left) and Chain Drag (right)

Recommendation

The sounding method should not be used on asphalt overlaid concrete decks.

Chain Drag Sounding

Chain drag is another way of finding delaminations and voids. Compared to sounding with a hammer, chain drag can cover more area in a given time. In this method, the operator passes a heavy chain on the surface of the concrete. The quality of sound generated is picked up using microphones and can be characterized by wavelengths. A schematic of chain drag was shown in the right side of Figure 5.13.

To conduct a sounding test, a grid is first laid on the concrete surface. Then the surface is sounded; the areas where delaminations are heard are recorded. A delamination map is then drawn and the amount of delamination is computed as a percentage of the total surface area. Spalls are not included. For bridge decks, 5-foot grid may be used. For substructure and/or superstructure members, the grid size may be 2.5-foot.

Suggested Procedure

Sound Testing

1. A grid is first laid on the concrete surface.
2. Then the surface is sounded; the areas where delaminations are heard are recorded.
3. A delamination map is then drawn and the amount of delamination is computed as a percentage of the total surface area. Spalls are not included. For bridge decks, 5-foot grid may be used. For substructure and/or superstructure members, the grid size may be 2.5-foot.

When the method is used on decks overlaid with cement concrete mixtures, it will detect both debonding of the overlay and delamination of the underlying concrete; but the method cannot distinguish debonding from delamination. A supplementary corrosion detection test may be used to delineate debonding from delamination. Note that delamination will be associated with bar corrosion, whereas debonding will not. The sounding method is considerably less accurate when used on decks overlaid with asphalt concrete and should not be used on decks overlaid with asphalt concrete. For more information, see ASTM D4580 *Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding*, 2012.

5.5.3.3 Corrosion Survey

Corrosion Potential

This procedure determines the potential for the existence of reinforcing steel corrosion through measuring the electrical potential of the reinforcing steel. The electrical measurements are made by placing a half-cell probe on the deck surface at predetermined grid points. The electrode may be attached to a hand-held voltmeter or it may be in the wheel of a rolling device that stores data for transfer to a laptop computer. Either way, the probe is attached to a rebar in the deck, which is in contact with the rebar mat being tested. A schematic of half-cell equipment for measuring corrosion potential is shown in Figure 5.14.

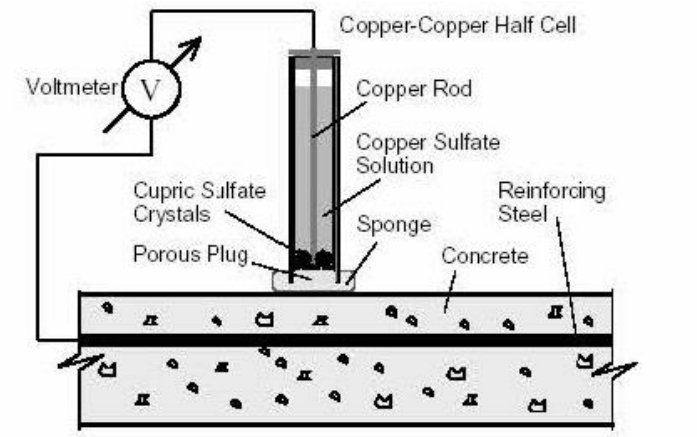


Figure 5.14 Schematic of Half-Cell Potential Test

An example of half-cell test equipment applied on a bridge deck is shown in Figure 5.15.

Figure 5.15 Example of Half-Cell Potential Test

The magnitude of the voltage measured determines the state of the steel with respect to corrosion. Laboratory tests have found that steel is corroding if the electrical potential is numerically greater than 0.35 Volts (relative to copper/copper sulfate reference cell) and steel is not corroding if the potential is numerically less than 0.20 Volts. For potential in the range of 0.20 to 0.35 Volts, the corrosion activity of steel was found to be uncertain. Electrical potential test results may be used for repair of reinforced concrete bridge elements. To do this, the deteriorated areas as well as all areas with a corrosion potential numerically more than 0.25 Volts (relative to copper/copper sulfate half-cell) are removed and replaced.

The half-cell corrosion detection test cannot be conducted when a dielectric material, such as a waterproofing membrane or polymer, is applied on the concrete surface, unless the material is punctured at the test location. Also, the test is not recommended for epoxy-coated reinforcement. This is because epoxy-coated bars are electrically insulated from each other. Older decks in which the epoxy coating is more deteriorated may be an exception; in that situation, half-cell testing may provide value.

Recommendation

Half-cell corrosion detection should not be used if the following conditions are present:

- The concrete surface is protected with polymer or waterproofing membrane
- Epoxy Coated reinforcement is present in the concrete member to be evaluated

Depth of Cover Meter (Pachometer)

The less concrete over the rebar, the less protection there is against salt penetration, since moisture can access the rebar more quickly. Cover depth may be measured by a cover meter (pachometer), which is a magnetic flux device. This device uses a magnetic field to detect reinforcing steel within concrete. It can determine the location, orientation, size, and depth of the bar. For a meaningful evaluation, a number of locations should be tested. An example of an appropriate number of tests is the greater of (1) 40 locations per member or (2) 40 locations per every 5,000 square feet of deck. Examples of a cover meter (pachometer) are shown in Figure 5.16 and Figure 5.17.

Figure 5.16 Pachometer in Use

Figure 5.17 Examples of Pachometer Equipment

The accuracy of the device in measuring the depth of cover decreases as the depth of cover increases. Magnetic particles in the concrete can influence the measurements. Thus, a correction factor should be obtained by exposing the bar at one location and measuring the actual depth. Usually, the correction factor is obtained at a location which can later serve as the half-cell test bar ground connection.

If the bar size is not known, due to equipment limitations using magnetic field measurements, the depth cannot be read directly from the scale. Therefore, the following procedure is suggested:

Suggested Procedure

Using a Pachometer

1. Locate the bar in the test area.
2. Place a two-by-four, or other non-metallic spacer, between the probe and the concrete surface.
3. Record possible bar size and depth combinations.
4. Correct readings for the thickness of the spacer by subtracting its thickness.
5. Place the probe directly on the concrete surface and record possible reading.

The bar size for which the same cover depth is obtained in Steps 4 and 5 is the correct result.

Cover depth may also be determined using Ground Penetrating Radar (GPR).

Corrosion Rate Measurement

Since concrete deck deterioration from salt contamination is one of the most expensive bridge maintenance problems, there are ongoing advancements in detecting and predicting the problem. Examples of newer, and perhaps more precise, methods of predicting corrosion damage are devices that measure the rate of corrosion and the permeability of concrete.

The corrosion rate device determines the rate of corrosion of reinforcing steel, or stated differently, the thickness of the rebar section loss every year. Corrosion current is expressed in terms of milliamperes per square foot of reinforcing steel area, which can be converted to section loss per year. These types of measurements may be performed using a field machine, together with appropriate probes and electrodes. An example field machine is shown in Figure 5.18. Also shown is a schematic of a typical probe and electrode set-up. Wire cable connections between field corrosion rate device and electrodes are not shown. Similar to a half-cell potential test, corrosion rate tests cannot be carried out where epoxy-coated or galvanized reinforcement is used or where there is a dielectric overlay or coating on the concrete.

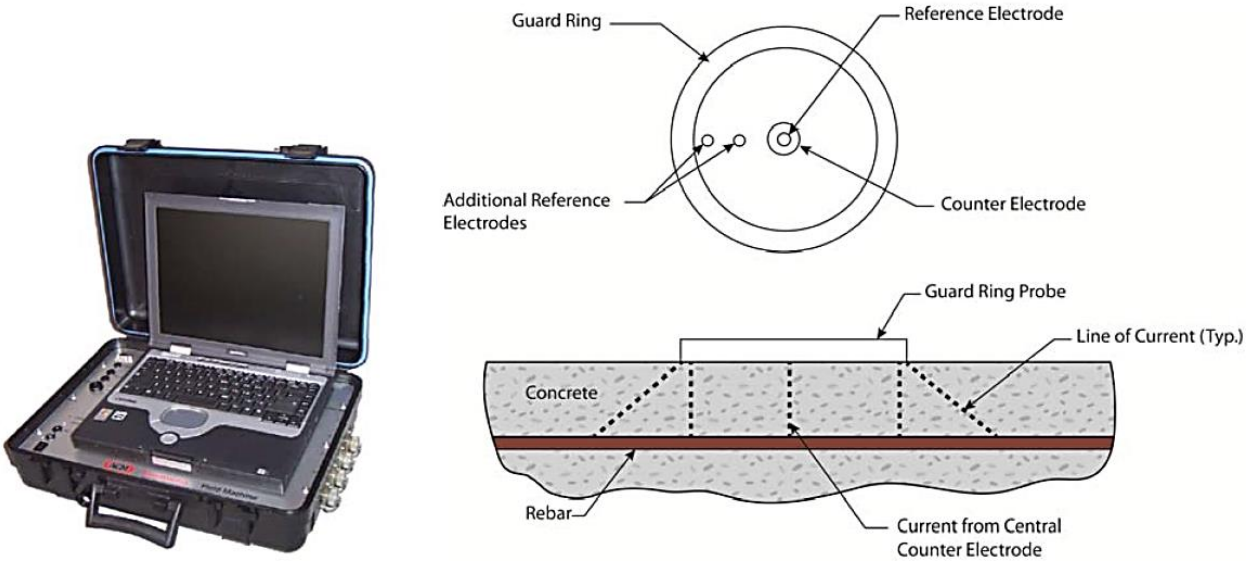


Figure 5.18 Example of a Corrosion Rate Device and Associated Electrode Schematic

5.5.3.4 Swiss Hammer for Approximate Compressive Strength

A hand held, spring loaded, device known as a Swiss Hammer (also called a Schmidt Hammer) can be used to gauge concrete compressive strength. Striking the spring tip against a concrete surface produces a gauge reading on the hammer that indicates the compressive strength. It is best to try this several times and take an average reading. The hammer is less accurate than laboratory compressive testing, but may be useful as a triage device in deciding whether to take cores and send them to a lab. A photo of a Swiss Hammer, or Schmidt Hammer device, is presented in Figure 5.19.

Figure 5.19 Swiss (Schmidt) Hammer

5.5.3.5 Portable Surface-Air Flow Device for Relative Permeability

Strategic Highway Research Program (SHRP) has developed a test procedure to measure relative concrete permeability non-destructively in the field. Using a portable surface-air flow device, the permeability measuring method creates a vacuum and measures the rate of air flow through the top half an inch of concrete. The higher the air flow rate, the higher will be the permeability. The SHRP method saves time by making on-site, non-destructive testing possible. The SHRP test method, *Method for Field Measurement of Concrete Permeability*, is documented in SHRP Report No. SHRP-S/FR-92-109, Strategic Highway Research Program, National Research Council, 1992.

5.5.3.6 Advanced NDT Techniques

Unlike a visual or hammer sounding where the assessment is a subjective determination, these techniques use data recorders and sensors to generate numbers.

Some of the more specialized methods appropriate for the evaluation of highway structures include: Impact echo (IE), Ultrasonic pulse echo (UPE), Ultrasonic surface waves (USW), Ground penetrating radar (GPR) and Infrared thermography (IT).

Non-destructive testing (NDT) of concrete is complicated because concrete is composed of aggregates, cement, rebar, and small air voids. It becomes difficult to distinguish between defects and an indication (a defect within tolerance is called an indication) in normally occurring situations within the concrete. In addition, a uniform pass/fail defect criteria for good concrete is not available.

Detailed information on the following advanced NDT Techniques is provided at:
http://www.ndtoolbox.org/

Impact Echo (IE)

The IE method is based on the reflection measurement of mechanical stress waves, i.e., the time it takes for the stress wave generated by an impact to be received by the receiver. This method was originally developed to determine the thickness of concrete slabs and to detect defects such as delamination and void within concrete structures and has been standardized

under ASTM C1383. The method has also been used for detection of debonding beneath Asphalt Concrete (AC) and Portland Cement Concrete (PCC) overlays. An example of the IE method is shown in Figure 5.20.

Recent advances in IE technology include the development of scanning systems and advanced signal processing approaches to speed up field testing and facilitate the interpretation of IE data. The IE method, with both point-by-point and scanning devices, can be used for characterizing delamination, overlay debonding, and for evaluation of vertical cracks.

Figure 5.20 Impact Echo Testing

Ultrasonic Pulse Echo (UPE)

A UPE test involves measuring the time it takes a high frequency ultrasonic wave to travel through a material between two or more points. Based on the time or velocity, this technique can also be used to detect the presence of internal flaws such as cracks, voids, delamination and other damage. Examples of ultrasonic pulse echo equipment are shown in Figure 5.21 and Figure 5.22.

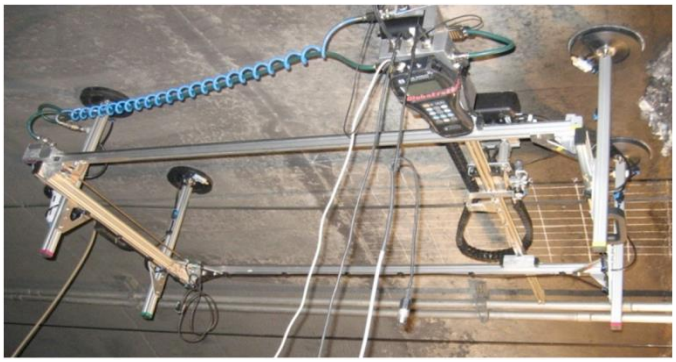


Figure 5.21 Ultrasonic Echo Control Unit Mounted on Automated Scanning Device

Figure 5.22 Ultrasonic Echo Control Unit and Sensor Array

Ultrasonic Surface Wave (USW)

With suitable equipment and processing software, an USW test can be rapidly performed to assess the properties of concrete including the changes in modulus caused by mix design, compaction, curing, stress-related cracking and concrete deterioration. For concrete deck deteriorations such as debonding and delamination, the USW method can play a supplemental role. Examples of USW equipment are shown in Figure 5.23 and Figure 5.24.

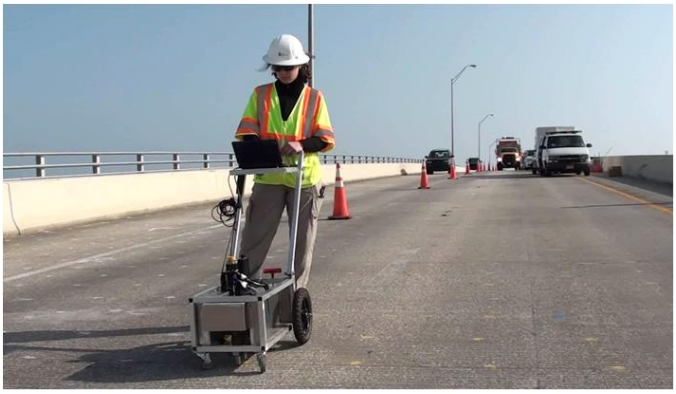


Figure 5.23 USW on Bridge Deck

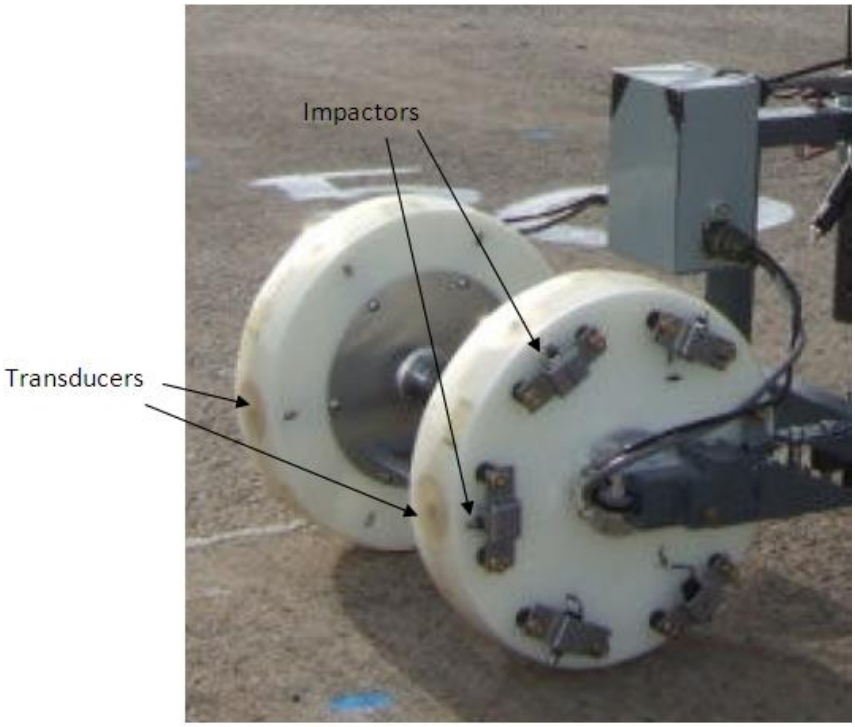


Figure 5.24 Bridge Deck Scanner Device has Two Identical Transducer Wheels

Ground Penetrating Radar (GPR)

The Ground Penetrating Radar (GPR) method is an electromagnetic-based reflection method that can provide graphical images of subsurface features. The images are useful for interpreting subsurface structure and layout. Application of this technique is good for detecting rebar location, pavement thickness, asphalt overlay thickness, and voids beneath pavements. A photo of a worker performing GPR testing is shown in Figure 5.25.

Figure 5.25 Worker Performing GPR Testing

GPR has a distinct advantage over other NDE methods since it is able to be used fairly quickly and easily for a variety of bridge decks and other reinforced concrete applications. The GPR

condition assessment reaches the top rebar mat and can be performed at highway speeds. An example of GPR results is shown in Figure 5.26.

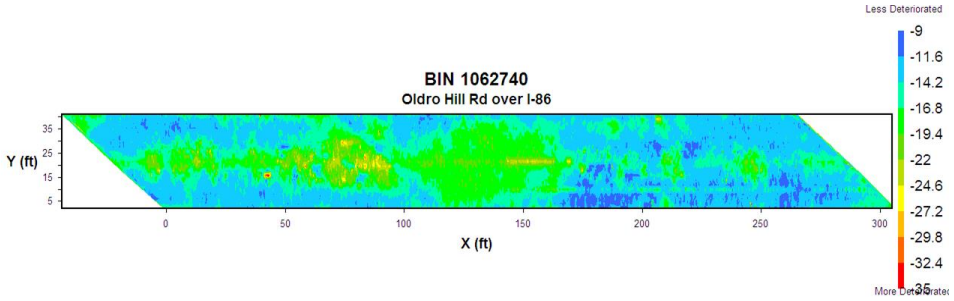


Figure 5.26. Map of GPR Results

Infrared Thermography (IT)

The detection of heat flow through a body can indicate the presence of flaws or defects. In the Infrared Thermography (IT) technique, heat is passed through the material, and an infrared detector detects the heat patterns flowing in the body. As shown in the schematic in Figure 5.27, when a defect is present in the body, it would show up as a hot spot when heat is flowing inward, and as a cold spot when the heat is flowing outward.

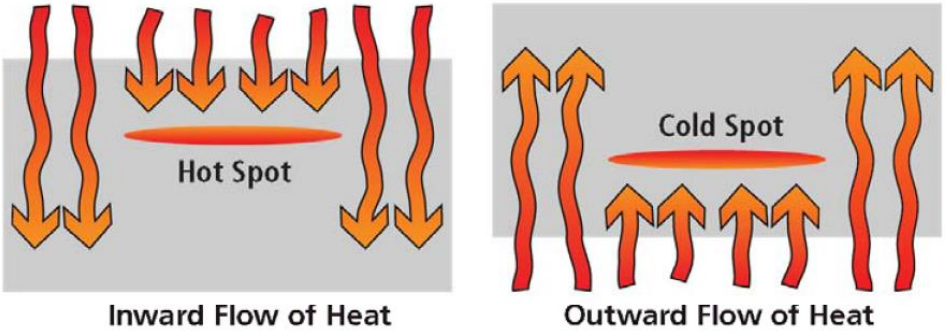
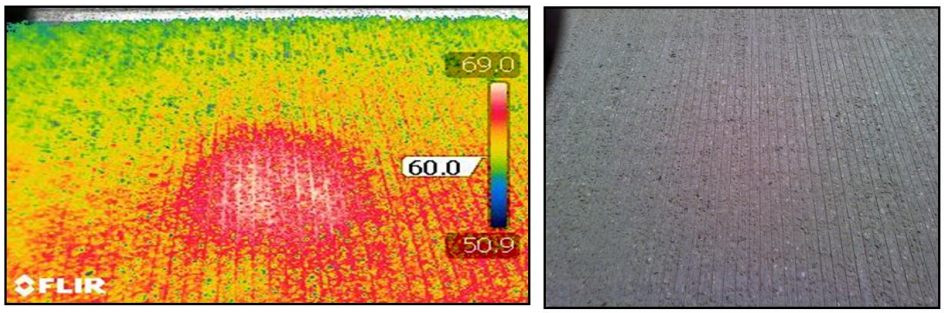


Figure 5.27 Schematic of Flow Direction and Corresponding Defect Appearance During IT

This technique is very useful in detecting delaminations in bridge decks. Complete thermal scan of the bridge deck may be obtained to identify defective areas. The limitations of this technique are that its accuracy is somewhat limited to the near-surface areas, and the application requires the presence of good weather and the suitable time of day. Also, surface conditions might have a bearing on the result. An example of IT results showing a subsurface defect and corresponding photo of the surface is presented in Figure 5.28.



(a) (b)

Figure 5.28 Example of (a) IT Results Showing Subsurface Defect (b) Corresponding Photo

The surface of the concrete over the delaminated area exhibits a temperature gradient (positive during the day and negative at night) with respect to its surrounding intact area. The capability and effectiveness of this method depend on climate conditions, as shown in Table 5.4.

Table 5.4 Environmental Conditions for Deck IT Inspections According to ASTM D 4788-07

Time of Year	Direct Sunlight (hours)	Change in Ambient Temp.	Wind Speed (miles per hour or mph)	Detection Limit (ΔT)	Notes
Summer	3	None specified	< 30 mph	1.8 °F	Dry deck for 24 hours prior to test
Winter	4	20 °F	< 15 mph	1.8 °F	Ambient temp. > 32 °F
Winter, asphalt	6	20 °F	< 15 mph	1.8 °F	Ambient temp. > 32 °F

The advantages of using thermography are:

- Real-time feedback for assessment while crews are mobilized
- No processing software needed; straightforward data analysis, works independently of PC
- Early detection of delaminations possible before visual signs of distress occur
- Hands-on access not required
- Relatively simple to use, essentially the same operation as consumer digital camera, whereas the investigation of very shallow areas in concrete structures often difficult with other NDT methods
- Testing at highway speeds possible

Disadvantages/Limitations:

- Favorable weather conditions are needed, i.e., clear skies, dry and calm winds.
- Generally the imaging is limited of a depth of 2 inches.

The limitations and scope of these techniques must be well understood. The type of test to be carried out and the extent of the investigation (i.e., whether only specific areas are to be selected or the entire structure needs investigation) should be properly detailed.

5.5.4 Destructive Testing

5.5.4.1 Concrete Cores

Cores are taken to provide a sample of the hardened concrete in place. Cores are used to perform microscopic analysis and other laboratory procedures that identify problems related to aggregate quality, air content, materials incompatibility or chemical contamination. Cores can also be tested to measure the compressive strength of the concrete. However, most concrete problems on existing bridges are related to durability rather than strength, so cores are rarely needed for strength.

The core sample technique requires drill equipment and subsequent laboratory testing. The method is destructive in that it causes micro-cracks and leaves holes in the base material, as shown in Figure 5.29. Since coring is moderately expensive and destructive, cores are typically taken only when other evidence indicates that further investigation is warranted. All core holes should be filled upon completion of coring with a suitable patching material. If the cores are taken on bridge decks, the material should gain at least 2000 psi strength or minimum mandated by the highway agency which owns the bridge before being exposed to traffic. Additional coring photos are included in Figure 5.30 and Figure 5.31.

Figure 5.29 Example of Core Removal

Figure 5.30 Core of Repaired Concrete at Perimeter of Patch

Figure 5.31 Coring Rig Anchored to Trailer Hitch

Petrographic UV Analysis

Once a core is taken to the laboratory it can analyzed for multiple issues including water and soluble chloride content, concrete mix type and concrete deterioration mechanisms such as ASR. From the core, a specimen is cut using a table saw, chemically cleaned, polish, etched, and placed under a microscope. An ultraviolet (UV) lamp is used to illuminate the surface of the sample and built in cameras record results. Under the UV light, the presence of the swelling gel associated with ASR can be easily seen, as shown in Figure 5.5.

Permeability

The durability of concrete is related to its permeability. Low permeability concrete is less susceptible to deterioration caused by environmental factors such as deicer corrosion, freezing-thawing, or alkali-silica reactivity. Permeability testing is performed on concrete core samples and determines the relative permeability of the concrete (or concrete overlay). Many highway agencies are currently using "Rapid Determination of the Chloride Permeability of Concrete," (AASHTO T277 / ASTM C1202) which requires coring a 4-inch diameter, 2-inch thick concrete and conducting a relatively simple laboratory test. The permeability is indicated by the electrical charge passed through the cored concrete. The electrical charge is expressed in terms

of coulombs. The higher the electrical charge passed, the higher is the permeability. Electrical charges less than 2000 coulombs represent low permeability. Those between 2000 and 4000 coulombs represent moderate permeability, and charges more than 4000 coulombs are indication of high permeability.

Performing field permeability testing was discussed in Section 5.5.3.5. An advantage of field testing is that no cores are taken, i.e., the test is performed on the concrete surface. However, field trials have shown that while an indicator of relative permeability, cores should be obtained and lab tested if more accurate values are needed.

Chloride Profile

This test determines the chloride (salt) content of concrete. Chlorides in concrete are the main reason for corrosion of the rebar and delamination and spalling of concrete. If water soluble chloride content of concrete exceeds 1.5 pounds/cubic yard, the potential for corrosion of the rebar exists.

Concrete samples are taken from the dust produced directly from the concrete by means of a rotary hammer and collected with a vacuum bit. The same hole is used to collect samples at different depths. Each time a sample is collected, the remaining residue is cleaned out before drilling can continue. For a meaningful evaluation a number of location should be sampled. An example of an appropriate number of test locations is the greater of 10 locations per element or 10 locations per every 5,000 square feet of deck.

Each of the powder samples is analyzed and the chloride content is graphed. Typically, the chloride content is higher near the surface and diminishes with depth. Examples of a chloride profile and chloride profile sampling equipment are shown in **Figure** 5.32 and Figure 5.33.

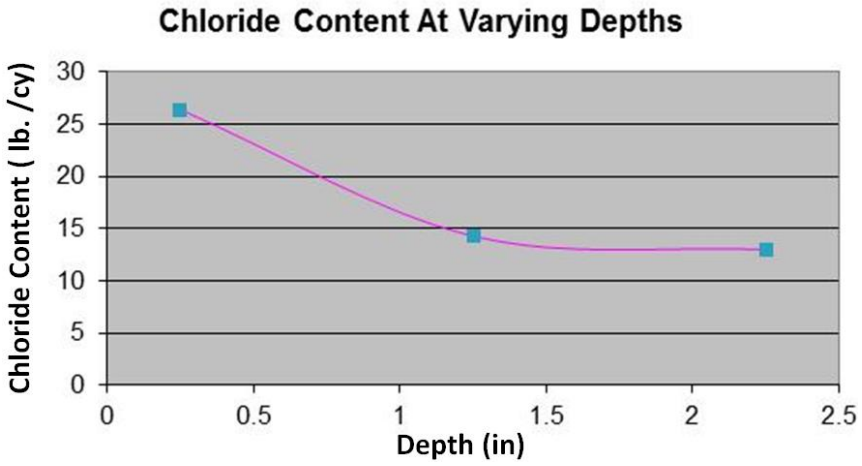


Figure 5.32 Example Graph of Chloride Content in Concrete Core

Figure 5.33 Chloride Profile Sample Collection

5.5.4.2 Endoscopes/Borescopes

Endoscopes (also known as borescopes) are viewing tubes that can be inserted into drilled holes for the inspection of hollow members such as concrete box girders and post-tensioning ducts. This could be considered a destructive test because it involves destroying a small portion of the member in order to insert the scope. The portable video endoscope/borescope provides high-resolution video images that are rich in detail and quality. They allow the user to capture images and video files of remote visual inspections for future reference. They are an excellent tool for inspection of difficult to see areas of a bridge such as behind bearings or anchorages, inside closed structural elements, etc. Figure 5.34 is a photograph of a typical endoscope/borescope with video capability.

Figure 5.34 Typical Endoscope/Borescope

5.6 Removal of Deteriorated Concrete

5.6.1 Introduction

Concrete can fail for a variety of reasons, such as from exposure conditions, excessive loads, infiltration of deleterious chemicals (de-icing salts, improper placement, and improper mix design). Lack of quality control and quality assurance procedures, inadequate training, limited knowledge of the materials, and poor aggregate are additional factors. An example of a severely deteriorated concrete pier is presented in Figure 5.35.

Figure 5.35 Severely Deteriorated Concrete Pier

The extent of deteriorated concrete removal operations on a given bridge component can vary from large regular areas covering most of the component to small irregular areas in random locations. Bridge decks present large surface areas that are exposed almost uniformly to deicing chemicals and water. This means that deteriorated areas and contaminated areas with potential for deterioration are more likely to occur on decks with almost a systematic pattern. Other bridge elements normally present smaller and irregular deteriorated surface areas. For example, concrete pier caps below leaking joints are also frequently deteriorated by drainage from above. Additionally, deterioration of bridge pier columns located near roadways is typically more prominent on the side subjected to salt laden spray from passing vehicles.

The depth of concrete removed has a profound effect on the method and cost of concrete removal. The necessary depth of concrete removal is tied to the extent of deterioration. Well bonded but stained concrete or concrete preparation to receive a sealer are situations that require only surface removal, such as sand or shot blasting. In contrast, spalled or delaminated concrete needs to be removed to a deeper level. This typically would require concrete removal from around the total perimeter of the rebar to obtain a good bond between the replacement concrete and reinforcement steel. If the rebars are corroded, then the rebars must be sandblasted from behind to arrest the corrosion.

5.6.2 Surface Abrasion and Cleaning

Often it is necessary to remove concrete surface contaminants, such as dirt, oil, and rubber on bridge decks, to provide a durable bond between the concrete surface and certain types of protective layer, such as a sealers and coatings, applied on the surface. The objective is to clean and provide a rough texture for bonding rather than to remove material. The two most frequently used methods for concrete surface removal and cleaning are sandblasting (abrasive blast cleaning) and shotblasting. Other options, such as sponge blasting and water blasting (either with abrasives or without abrasives in the water) are also effective in abrading concrete surfaces.

5.6.3 Abrasive Blast Cleaning

Abrasive blast cleaning uses compressed air to propel particles at high velocity onto the surface. The impact of particles produces a very abrasive action which cleans and roughens the exposed concrete. Sand and coal slag (e.g., Black Beauty) are commonly used to abrade the concrete surface. As the blast particles break down contaminates are released into the air and can be inhaled by workers and residents in the area. Recycled glass and very hard aggregates such as garnet do not break down as completely and are safer to use as abrasives, are more environmentally friendly, and perform as well or better than sand or coal slag. Examples of abrasive blast cleaning are shown in Figure 5.36 and Figure 5.37.

Figure 5.36 Abrasive Blast Cleaning Along the Curb Line

Figure 5.37 Abrasive Blast Cleaning a Concrete Barrier

As the concrete surface is abrasive blast cleaned, dust is created. The dust contains silica and very small particles that can cause respiratory and other health related problems. Workers should always wear proper respiratory protection.

Recommendation

Always wear proper respiratory protection when abrasive blast cleaning.

5.6.4 Shotblasting

Shotblasters propel steel shot at high velocity against the concrete surface. The impact is capable of removing concrete to depths up to 1/4 inch. The rate at which the machine moves across the surface effects the amount of concrete removed. Machines vary from ride-on types to walk-behind and vary in widths of cut. Shotblasting can also be performed manually. Shotblasting is typically used to prepare the bridge deck surface for a rigid concrete or polymer overlay. An example of shotblasting is shown in Figure 5.38.

Figure 5.38 Shotblasting a Bridge Deck

A collection chamber collects debris and concrete dust and steel shot. The shot is separated from the debris and recycled. The debris should be disposed of as an industrial waste. A schematic describing the shotblasting process is presented in Figure 5.39.

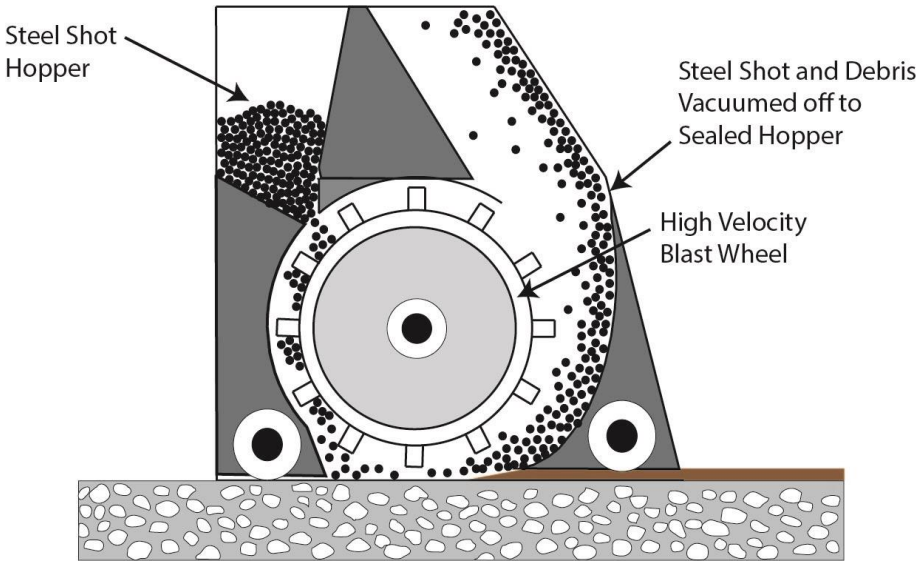


Figure 5.39 Typical Shotblasting Blast Wheel

5.6.5 Cover Removal

Certain types of protective layer, such as bridge deck concrete overlays, require substantial roughening of the surfaces for bond. This and the fact that these systems add to the deadload substantially require partial removal of concrete cover to reduce the dead load and maintain the live load carrying capacity of the structure at the pre-repair level. The work involves removal to a depth less than the cover depth of the steel and thus no work between, around, or under the reinforcing mat is included. Cover removal is normally done by a milling machine, also known as a scarifier, or hydrodemolition, or a combination of both. The machines remove concrete by the impact of numerous teeth mounted on a rotating drum (milling) or pressurized water (hydrodemolition. These techniques are typically used for concrete removal on horizontal surfaces. Refer to Section 5.6.7.2 for additional information on hydrodemolition. Severe damage to reinforcement can occur when milling teeth cut or snag the reinforcement.

Therefore, the teeth should be kept clear from the reinforcement or other metallic objects on the surface. The final step is blast cleaning of the concrete surface (e.g., shotblast, abrasive blast, or waterblast) to remove loose particles.

5.6.6 Saw Cutting

The recommended procedure to prepare the patch area is to saw cut the perimeter of the patch approximately 1/2 inch to 1 inch deep. Make square cuts. If a concrete overlay is to be applied monolithic with the repair, saw cutting is unnecessary and the edge of the areas of removal should be chipped at about 45 degrees to prevent pockets of entrapped air when placing the overlay. Feather-edging the patching material should always be avoided. Sharp edges, at least 1 inch deep should be formed by jackhammers (not preferred) or, preferably, by saw cutting. When saw cutting, it is advantageous to tilt the saw blade to key in the patch by making it wider at the bottom than at the deck surface. This can be done by running one wheel of the saw on a plank placed on the deck. This angled portion would be made below the 1/2 inch to 1 inch deep vertical surface saw cut. An example of saw cutting and removal of deck cover concrete is shown in Figure 5.40.

Figure 5.40 Saw cut and Removal of Deck Cover Concrete

5.6.7 Removal around Reinforcing Steel

When spalls, delaminations, and contamination have progressed into the concrete that surrounds the reinforcing steel, it is necessary to remove concrete from between and below the reinforcing steel in addition to removing the cover. An example of saw cut removal around reinforcing steel is shown in Figure 5.41. The work must be done such that the steel is not damaged. Also, in the adjacent areas where the concrete is not removed, the bond between steel and concrete must not be destroyed, and the substrate concrete must not be cracked.

Figure 5.41 Saw cut Around Reinforcing Steel with Concrete Removed on an Angle

The two common methods to remove concrete without damaging the rebar are pneumatic breakers (jack-hammers) and hydrodemolition (a form of water blasting). Both of these systems can be used to remove concrete cover and concrete from between and below reinforcing steel. They could also be used to remove concrete from between and below reinforcing steel after a more high production method, such as milling, has been used to remove the cover concrete . If concrete has an asphalt overlay, it should be milled prior to hydrodemolition. Hydrodemolition can be used without milling if delaminations or spalls are present.

Jack hammers are suited for small jobs. However, improper handling of the hammer can destroy good concrete. Hydrodemolition is better for large jobs. It also does not cause any of the damage discussed above. Estimating the quantity of concrete to be removed prior to repairs is not an easy task, especially if it is intended that only unsound concrete be removed. Substantial overruns have not been uncommon. Estimating errors can be minimized by a thorough concrete condition survey as close as possible to the time the work is executed.

When the survey is done years prior to the repair, it is common to increase the estimated quantities by an arbitrary amount, usually 25 percent, to account for continued deterioration. Before concrete removal is started on a structural member, the member should be analyzed to determine whether shoring or formwork is required. After the concrete is removed, the remaining section must support its weight, any superimposed dead load, live load if the bridge is to be repaired under traffic, formwork, equipment, and the weight of plastic concrete. On a flexural member the final dead load deflection must be compatible with the other members in the unit.

When to Call the Engineer

When removing concrete around reinforcing steel, the Engineer should be contacted to determine if shoring or temporary support is necessary.

5.6.7.1 Pneumatic Breakers (Jack-Hammers)

The pneumatic breaker (commonly known as a jack-hammer) is the most prevalent method for removing concrete from between and below reinforcing steel. An example of a worker jack hammering concrete is shown in Figure 5.42. It is a hand-held tool and powered by compressed

air to deliver a series of high frequency blows that fracture concrete in a small, controlled area. The production depends on the size of the hammer and the skill of the operator.

Figure 5.42 Hammering Concrete

It is common practice to restrict the size of hammers used in concrete removal to prevent damage to otherwise sound concrete. A typical restriction is the use of a 30 pound maximum size jackhammer above the top reinforcing steel and a 15 pound maximum size chipping hammer between and below the reinforcing steel. The existing concrete is removed to sound concrete. Pneumatic hammers should be worked at an angle of 45 to 60 degrees to the plane of the surface being removed.

Some states require removal to below the top mat of reinforcing steel to ensure the new concrete is “locked-in” by the rebar, but practices vary widely. The concrete directly below the top mat (i.e., the “shadow area”) may be in good condition and care must be taken not to hit the rebar with the hammer as this will damage the rebar and may cause cracking in the concrete that is to remain. Figure 5.43 shows the top mat of reinforcement exposed and ready to be patched.

Figure 5.43 Extensive Patching on a Bridge Deck

Where it is necessary to chip below the bar, a clear space of 1/4 inch plus the maximum size of the aggregate to be used in the repair concrete must be provided. This space is typically 1 inch beyond the reinforcing bar. It is also usual to expose bars that are heavily rusted or where there

are heavy rust deposits in the concrete adjacent to the bar. The final step in preparing the existing concrete surface is blast cleaning of the concrete and the exposed steel. If the cause of deterioration is chloride-induced corrosion of the reinforcing steel, it is essential to remove all salt and chlorides from the reinforcement. Abrasive blast cleaning is typically done to remove rust and abrade the surface for the new concrete. After completion of this operation, the concrete should be carefully inspected and aggregate particles that have been cracked or fractured during chipping should be removed to a sound surface.

In areas of very badly deteriorated concrete decks, full-depth removal may be necessary. In such cases, forms must be attached to the soffit of the deck.

5.6.7.2 Hydrodemolition

Hydrodemolition is a high-production method for concrete removal. An extremely high-pressure water jet is used to destroy the cement matrix and liberate aggregate. The hydrodemolition equipment is expensive and complex. This method of concrete removal needs special skills to produce the desired results. Although hydrodemolition work is mainly used to remove concrete from between and below the reinforcing steel, the equipment can be calibrated to remove concrete to almost any depth. The hydrodemolition process is such that the system self-adjusts the depth of concrete removal depending on the soundness of the material encountered. An example of an area treated by hydrodemolition is shown in Figure 5.44.

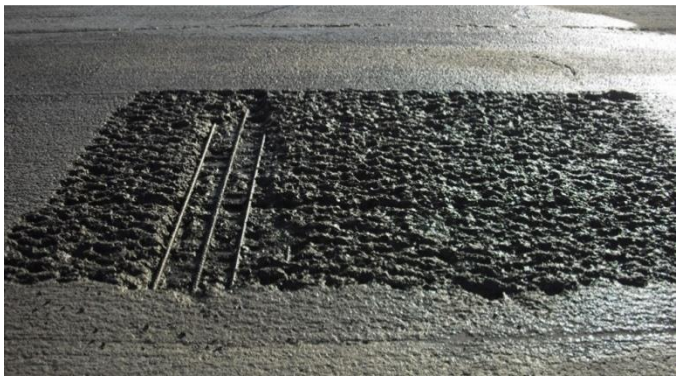


Figure 5.44 Hydrodemolition Test Section

Hydrodemolition is mainly used on horizontal surfaces such as bridge decks and for large-scale concrete removal to minimize cost. However, the method can be used on inclined, vertical, and overhead surfaces but cost effectiveness is reduced by the excessive cost of the specialized equipment needed to safely direct the jet and contain the debris.

Hydrodemolition works well on decks with uniform concrete. Decks that have a variety of patches of differing strength or hardness of concrete may not yield uniform removal depths when hydrodemolition is used.

5.6.8 Concrete Removal on Pier Caps and Columns

Because of leaky joints, bridge piers may have significant amounts of concrete deterioration which would require extensive concrete removal. It is imperative that the structural integrity of the piers be maintained during the repair process. Guidelines limiting the amount of concrete removal that can be completed at any one time on the pier/bent cap, beams and columns

should be adhered. Techniques for concrete removal on pier caps and columns are very similar to those discussed above for bridge decks.

When to Call the Engineer

A structural engineer should be consulted to ensure that a sufficient number of rebars remain encased or in contact with the existing concrete during the rehab process. If so much of the concrete around reinforcement is deteriorated and has to come out, the structural engineer can design temporary supports until the repairs can be made.

The single bottom mat of reinforcing steel in the pier caps provide the majority of the tensile strength in the pier caps between the columns. In the columns, the reinforcing steel provides resistance to axial loads and bending and shear stability for the concrete.

5.7 Replacement of Deteriorated Concrete

Once the unsound concrete is removed, the replacement concrete can be placed. A term used in bridge maintenance to replace unsound concrete in an existing structure is to 'patch' the concrete. A patch is often thought of as a temporary fix, but that can be subjective and dependent on the quality of the patch. If most of the procedures in this manual are followed, many owners consider the patch as a permanent repair. Examples of a temporary patch include ones where saw cutting or hammering was not used, or asphalt or a concrete of non-similar quality of the original material was used. Some owners consider any localized patch as temporary, but a larger repair like a deck overlay as permanent.

The following sections address Estimating, Purchasing, Delivering, and Storing Concrete Repair Materials. Specific references are provided at the end of this chapter.

5.7.1 Estimating

To estimate the quantity of concrete repair material required for a job, calculate the volume of the cavity to be filled in cubic feet. Divide the calculated volume by the yield in cubic feet of one unit (sometime called kits) of concrete repair material to calculate how many units will be needed. Add 5 percent to 10 percent for waste and spillage. Estimate the waste higher for smaller jobs.

The most common cavity shape encountered in repair operations is a rectangle. When the shape of the area to be repaired is irregular or uneven in depth, the average length, width, and depth must be determined by measurement or by estimate. Consider that an irregular shape should be squared off to help make a more durable repair.

To find the volume (V) of a rectangle in cubic feet (for example a pothole) multiply the length (L) in inches times the width (W) also in inches times the depth (D) also in inches.

$$V = L \times W \times D$$

The calculated volume will be in cubic inches. As most repair products are sold in cubic feet, it is necessary to divide the cubic inches by 1728. This is because 1728 is the number of cubic inches

in one cubic foot. Alternatively, if the estimation is performed in yards, divide the cubic feet by 27.

5.7.2 Purchasing

To compare the cost of a concrete repair material needed for a job, determine the cost per cubic foot for each material under consideration. Because concrete repair materials are sold in different sized bags or pails, or may yield different volumes when extended, it is necessary to compare cost on volume. Following this procedure will provide a basis for cost comparison. Comparing the number of units is not an accurate method.

5.7.3 Delivery

Concrete repair materials should be delivered, premeasured, and packaged in sealed, dry containers, and on shrink-wrapped pallets to prevent shipping damage. Repair material may also be delivered in ready mix trucks.

5.7.4 Storage

Concrete repair materials should be stored in a dry location at a temperature near 70 °F degrees, but never below 40 °F or above 90 °F. Concrete repair materials that become damp or otherwise defective should not be used and should be removed from the job site. Concrete repair materials that are beyond the manufacturer expiration date should not be used unless the manufacturer has requalified them by actual laboratory testing.

Recommendation

Do not use concrete repair materials that are beyond the manufacturer expiration date unless the manufacturer has requalified them by laboratory testing.

5.7.5 Preparation of Reinforcement

Prior to replacing concrete, exposed reinforcing steel in the removal areas needs to be prepared for the repair. It is important that loose bars be tied at each intersection point, as shown Figure 5.45. Tying the reinforcing bars prevents relative movement of the bars and the concrete in the repair under the action of traffic in adjacent lanes during the curing period.

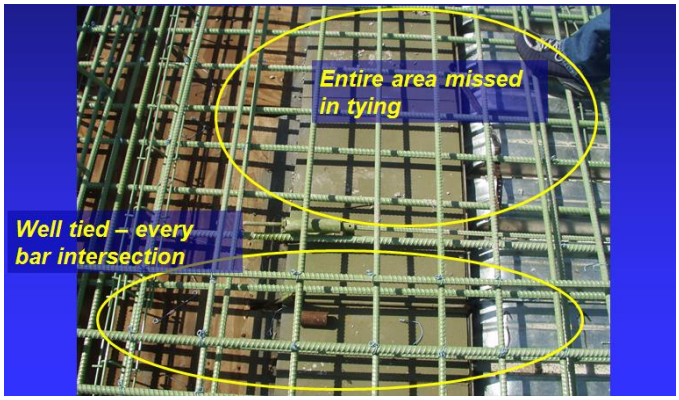


Figure 5.45 Reinforcement Bars Should be Tied at Intersection Points

If corrosion has reduced the cross-sectional area of the reinforcement by more than 20 percent, extra reinforcement is usually added. The typical method of adding reinforcement is to lap the weakened bars with additional bars to restore the cross-sectional area to its original value. The lap lengths over the undamaged parts of the bar should be the same as those required for new construction (typically 30 times the bar diameter). The lapped bars should be tied at both ends, as a minimum. If lapping is not possible, holes are drilled in the existing concrete and dowels or anchors installed. Mechanical splicers may also be used. The preparation of plain (not epoxy coated) reinforcing steel prior to replacement may include a calcium nitrate rust inhibitor coating of the reinforcement or use of sacrificial anodes (see Figure 5.46) to prevent further corrosion. The preparation of reinforcing steel prior to placement of repair material should include abrasive blast cleaning.

Figure 5.46 Sacrificial Anode Secured to Rebar

5.7.6 Types of Concrete Replacement

Areas of concrete removal can be repaired in several different ways, including:

- Form and Pour (Recasting) with concrete
- Prepacking dry aggregates and grouting
- Patching with cementitious, or polymer-based materials
- Shotcrete

On overview of each of these four methods will be presented in this chapter. Additionally, there are numerous American Concrete Institute (ACI) publications on repair techniques. The following repair manuals are available for free at the online bookstore for the American Concrete Institute http://www.concrete.org/Store.

RAP-1 Structural Crack Repair by Epoxy Injection
RAP-2 Crack Repair by Gravity Feed with Resin
RAP-3 Spall Repair by Low-Pressure Spraying
RAP-4 Surface Repair Using Form-and-Pour Techniques
RAP-5 Surface Repair Using Form-and-Pump Techniques
RAP-6 Vertical and Overhead Spall Repair by Hand Application
RAP-7 Spall Repair of Horizontal Concrete Surfaces
RAP-8 Installation of Embedded Galvanic Anodes
RAP-9 Spall Repair by the Preplaced Aggregate Method
RAP-10 Leveling and Reprofiling of Vertical and Overhead Surfaces
RAP-11 Slabjacking
RAP-12 Concrete Repair by Shotcrete Application
RAP-13 Methacrylate Flood Coat
RAP-14 Concrete Removal Using Hydrodemolition

Preparing Old Concrete for Repair with Cementitious Materials

To ensure an adequate bond and to reduce shrinkage of the new cementitious concrete, the existing concrete needs to be prepared. The first step, after the deteriorated concrete is removed, is to remove all loose particles, oil, grease, laitance, or any other material that would prevent the bonding to the concrete surface. The existing surface should be cleaned by light sand-blasting or other surface abrasion methods.

After cleaning, the concrete surface should be saturated with water spray, for 24 hours, if possible to create a saturated surface-dry condition. If a 24 hour saturation is not practical, the concrete should receive a water spray for as long as possible. A quick “blessing” of sprinkled water just before the pour will not pre-soak the existing concrete sufficiently. The mix water in the new concrete will be soaked up by the existing concrete. The result will lead to increased shrinkage cracks and poor bond.

Suggested Procedure

Steps for Achieving a Saturated Surface Dry (SSD) Condition

1. Clean concrete surface
2. Saturate repair area with water
3. Remove any standing water

Prior to the pour, any water remaining in the hole will need to be removed by waiting out or air blasting. If standing water remains, the quality of the new concrete will be affected. The compressive strength and durability of the repair will be compromised.

Next, a bonding agent (usually a cement grout or epoxy bonding agent) is well worked onto the concrete surface just before the installation of formwork. It is very important that the bonding

agent does not dry out when the repair concrete is recast. Bonding agents are not always required because if the agent dries out, it can actually become a bond breaker.

5.7.6.1 Form and Pour with Concrete (Recasting)

The form and pour method uses forms to repair bridge elements such as beams, columns, and wingwalls (see Figure 5.47 through Figure 5.50). If the formwork installation is lengthy, the use of bonding grout may not be possible, since it will dry out before the concrete is cast. In this case, specially formulated polymer bonding agents may be used which allow more working time.

Figure 5.47 Forming for a Wingwall Repair

Figure 5.48 Manufactured Forms for Columns

Figure 5.49 Forming for a Pier Cap Repair

Figure 5.50 Forming for Deck Repair

Formwork for recasting should be rigid enough to prevent new concrete from sagging away from the existing concrete under the weight of new concrete. The formwork should with stand forces from pumping of the concrete, if pumping is used to transport the concrete. Also, it should withstand the forces from the external vibrators, if such vibrators are used to consolidate the concrete.

Steel forms have the advantage of being rigid, but they are heavy and not easily handled. Another advantage of steel forms is that they allow heat dissipation of the concrete during the hydration. This will minimize thermal shrinkage and subsequent cracking of the new concrete. Forms are either supported by props or fixed to the structure itself. To repair a vertical patch, a “birds mouth” is built into the forms to allow the placement of concrete, as shown in Figure 5.51. Prior to placing the concrete, the forms should be cleaned, sprayed with a form release agent and wetted to prevent absorption of the water used in the concrete. Concrete for recasting should easily flow and fill all the voids in the form. Since space is limited, use of

internal vibrators is usually restricted. Ideally, the mix should be very workable; however use of water is limited to keep drying shrinkage to a minimum.

Figure 5.51 Birds Mouth Examples

Superplasticizers may be used to provide workability without resorting to water. Good compaction is achieved by placing the concrete in small amounts and vibrating effectively as the work proceeds. External vibrators are often used. When recasting deck soffit, sometimes it is possible to drill through the deck and place the concrete from the above. For difficult areas to access or in locations with considerable rebar, self-consolidating concrete could be used.

Consolidation

In some types of construction, the concrete is placed in forms, and then consolidated. Consolidation compacts fresh concrete to mold it within the forms and around embedded items and reinforcement and to eliminate stone pockets, honeycomb, and entrapped air. It should not remove significant amounts of intentionally entrained air. Vibration, either internal or external, is the most widely used method for consolidating concrete (see example in Figure 5.52). When concrete is vibrated, the internal friction between the aggregate particles is temporarily destroyed and the concrete behaves like a liquid; it settles in the forms under the action of gravity and the large entrapped air voids rise more easily to the surface. Internal friction is reestablished as soon as vibration stops.

An option to consolidation for these types of repairs has been self-consolidating concrete which was discussed earlier in this chapter. This type of concrete is very flowable and needs no vibration. Excess use of a vibrator can actually damage the composition of the concrete.

Figure 5.52 Pumping with Elephant Trunk and Consolidating with Vibrator

5.7.6.2 Prepacking Dry Aggregates and Grouting

The surface preparation and formwork for this technique is basically the same as those for the form and pour (recasting) method. The only difference is that dry aggregate is packed in the space behind the form so that it fills the space completely. Subsequently, grout is pumped from the lowest to the highest point to fill the space between the aggregate. Typically, a uniform size aggregate is used. An advantage of prepacking dry aggregate and grouting is that the overall shrinkage of the repair is eliminated.

5.7.6.3 Horizontal Patching

Horizontal patching does not require formwork unless the patch is a full depth type patch, for example on a bridge deck. These larger type patches are covered in Chapter 7. Small concrete areas up to 2 inches in depth can be patched with trowel-applied cement mortar regardless of the type of surface. To begin, clean the surface of the existing concrete by abrasive blast cleaning and bring it to a saturated surface dry condition. Subsequently, a conventional bonding agent may be used with caution as it can dry out (or prematurely set-up). Since no formwork is required, patching can be done immediately after the application of the bonding agent, thus preventing the bonding agent from drying out. The patch mortar should be firmly applied into place so that entrapped air is excluded as much as possible. The closer the physical properties of the patch material are to the existing material, the better. It is also important to minimize the shrinkage of the patching material. This can be done by using the exact amount of mixing water and ensuring the cement and water are well mixed. The inclusion of latex in a concrete or mortar will help reduce the amount of water required for workability and also reduce the permeability of the patch. Other performance-proven additives can be used to reduce setting time and increase strength. The aggregate can be adjusted in the mix design for thin patches.

Proper curing is important for all cement concrete. Thin patches present a particularly difficult problem because they dry out or cure quickly. The existing concrete will tend to absorb the moisture in the patching material. If exposed to the sun or wind, the moisture evaporates. It is recommended that thin patches be cured by covering with moist burlap and polyethylene sheet

on top. Without the polyethylene sheet on top, the burlap will dry out faster, doing more harm than good.

To protect the rebar from additional corrosion a corrosion inhibitor can be brushed onto the steel. Furthermore, galvanic anodes that contain zinc can be tied onto the rebar, as shown in Figure 5.53. The zinc contained within the anode will protect the steel from further corrosion.

Figure 5.53 Galvanic Anodes Placed into a Patch of a Pier Cap

Non-Portland Based Concretes

In addition to conventional cement based material, there are two types of materials that are used for patching concrete: quick-setting hydraulic cements and polymer materials.

There are a variety of proprietary brands of quick-set hydraulic and polymer material that are designed to gain strength rapidly. Each proprietary material has a number of unique properties favorable for bridge maintenance work. For some materials, the patch may be opened to bridge deck traffic after about one hour. Quick-setting hydraulic cements most commonly used are formed by the reaction between magnesium-oxide and a soluble phosphate. These materials have material properties that exceed those of Portland-based cements, but they are more expensive and require additional care during the mixing and installation. “Mag-Phosphates”, as they are commonly called, can reach a compressive strength of 2,100 psi in less than an hour. Ultimate strengths can reach 10,000 psi. These materials also have good adhesion properties and are marketed for vertical and overhead applications as well.

Care must be taken during the mixing not to deviate from the recommended amount of mixing water. Even a small deviation will severely impact the durability of the cement. An example of a failed rapid-setting mix is shown in Figure 5.54. Aggregates used to make the concrete should not be calcareous (containing limestone) as these aggregates react with the cement to form a gel that deteriorates the concrete in presence of water over time. The rapid hardening process creates considerably more heat during hydration than a Portland-based reaction. Failure to control the heat of hydration will cause severe shrinkage cracking. In order to limit the amount of heat generated, Mag-Phosphate cements should not be placed in lifts greater than 4”. Modified formulations have been developed for repairs done in hot weather.

Figure 5.54 Failure of Rapid-Setting Mix

Polymer Concrete

Several types of polymers, such as epoxy, polyesters, methylacrylates, and ureathanes, have been developed for the repair of concrete. Polymers consist of a resin and a hardener. These two components are proportioned and mixed at the site. Aggregated is added to improve the compressive strength and because polymers are expensive, reduce the amount of material used. The mix solidifies upon a rapid chemical reaction. Priming the existing surface with the polymer material (without aggregate) provides good bond. Polymer materials liberate heat, set, and gain strength very rapidly. Their pot life is very short, especially when they are used in large quantities or in hot weather. This is because more heat is liberated and this in turn expedites setting of the polymer. On the other hand, their strength gain usually slows down substantially when the air temperature is 65 °F or below, and they may not gain sufficient strength.

Each type of polymer has material characteristics that make them unique from cement. Two of the more commonly discussed are viscosity and elongation. Viscosity refers the ability of the material to flow. A very viscous material like molasses has a high viscosity number. Water is a low viscosity material. Polymer materials can be made fluid enough to flow into place by gravity so inaccessible places can be filled with them without compaction.

Aggregates for Polymer Concrete

Polymer materials expand and shrink much greater than concrete when ambient temperature fluctuates. This unfavorable characteristics of polymer materials will cause debonding and separation of the patch from the old concrete in time. The addition of aggregate helps to alleviate this problem.

Aggregates must be dry and free of fine particles. Moisture negatively affects the polymer reaction and additional polymer would be necessary if too many fines are included. A hard durable 3/8 inch sized aggregate is used in extending polymer concrete. A more angular aggregate produces a stronger matrix so that less polymer is needed.

Mixing Polymers

Each of the two parts, the resin and the hardener, should each be stirred separately for about 3 minutes or as per manufacturer recommendation prior to mixing together. This is necessary to

re-mix any settled particles. Polymers are mixed in a ratio, typically one part resin to one part hardener, though some polymers may be formulated differently. Care should be taken to ensure the entire amount of resin and hardener are removed from each container. These combined parts should be mixed again for 5 minutes before adding the aggregate. Add the aggregates to the polymer. This helps to ensure the aggregates are coated with the polymer and thereby a more durable patch. Mixing should continue for another 5 minutes. Be sure to closely follow the manufacturer instructions.

Recommendation

Always follow the manufacturer instructions when mixing polymers.

Examples of mixing for smaller patches and for larger patches are presented in Figure 5.55 and Figure 5.56, respectively.

Figure 5.55 Five Gallon Bucket Mixing Method for Small Patches

Figure 5.56 Power Mixers for Larger Patches

Placing Polymers

All surfaces should be dry and free of any contaminates. It is critical the surfaces are dry. Adding a primer, as shown in Figure 5.57, will allow the polymer to bond better to the existing concrete. Unlike cementitious-based repair materials, the bond capability of polymer-based materials is significantly reduced in the presence of water. The surfaces should be clean of loose materials, rust, and oil free. The use of an electric leaf blower to air clean surfaces is recommended, as shown in Figure 5.58. Unlike combustible motors driven air compressor that rely on water and oil filters to provide clean air, the air from electric leaf blowers will not contain these contaminates.

Figure 5.57 Priming a Patch with Polymer

Figure 5.58 Using a Leaf Blower to Clean Surfaces

For a neat looking repair, duct tape is used to protect the perimeter concrete from spilled polymer that becomes difficult to remove once hardened. An example is shown in Figure 5.59.

Figure 5.59 Taping the Perimeter for a Clean Look

As polymers are generally not an electrically conductive material, the use of sacrificial anodes to protect the rebar is ineffective.

5.7.6.4 Shotcrete

Shotcrete, or pneumatic applied mortar, is used to repair and restore the surface of concrete bridge elements. It is conveyed through a hose and nozzle and pneumatically projected at high velocity onto a surface. It contains the same cement, aggregate and water as concrete except that there are no course aggregates in the mix. The mix has high cement content and low water/cement ratio. Addition of silica fume, fly ash and/or slag enhances performance of shotcrete. Shotcrete can contain silica fume at a minimum of 7 percent by weight of cement, fly ash at a maximum of 20 percent and slag at a maximum 50 percent. Steel or synthetic fibers have also been used to increase compressive strength and decrease potential for cracking. When properly applied, mortar is dense, durable and strong. It is also reported to have superior bonding characteristics. A typical shotcrete machine is shown in Figure 5.60.

Figure 5.60 Typical Shotcrete Machine

The applicator controls the quality of the work. The applicator controls the amount of mix water that is added to the dry mix and the amount of material applied to the surface, as shown in Figure 5.61. A field verification jig is commonly used to assess the skill level of the applicator prior to performing the work. Qualification of applicators and test panel should be required.

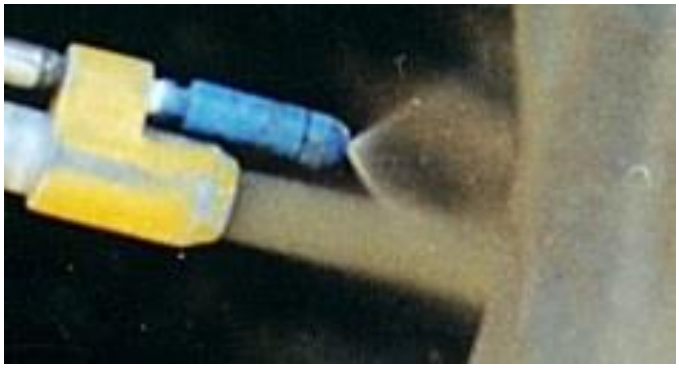


Figure 5.61 Mix Water Added at Nozzle

On vertical and overhead surfaces, since forming is not required, pneumatic mortar can be a useful alternative in bridge restoration. Large surface areas can be repaired in relatively short periods of time. Examples are shown in Figure 5.62 and Figure 5.63.

Figure 5.62 Shotcrete Repair of an Invert of a Corrugated Metal Culvert

Figure 5.63 Shotcrete Repair of a Pier

Preparation of the existing surface is an important part of the repair. All deteriorated concrete should be removed, as previously described in this manual, with the edges sloped about 45° to prevent entrapment of rebound material. Existing concrete and reinforcing steel should be cleaned by abrasive blast cleaning. Reinforcement of the patch with welded wire fabric or small diameter wire mesh is advised for depths exceeding 3 inches. The limits of the existing concrete face should be exceeded, if necessary, to obtain a minimum 2 inches cover on existing reinforcement.

Recommendation

Mortar should not be used in thicknesses of less than 1 inch.

Mortar should not be used in thicknesses of less than 1 inch. The existing surface is wetted as needed so it will not absorb water from the pneumatic mortar. The natural handgun finish is preferred from bond and durability standpoints. Scraping or cutting may be used to remove

high points and material that has exceeded the limits of the repair after the mortar has become still enough to withstand the pull of the cutting device. Troweling or other finishing is discouraged as it has a tendency to disturb the bond. Curing is very important for the rich mixes and thin sections used with pneumatic mortar. Seven days of water curing is generally advisable to promote good hydration of the cement, keep the mortar cool in hot weather, and prevent early shrinkage that may disturb the bond. Any defects in the shotcrete, such as the "Bug-holes" shown in Figure 5.64, should be repaired prior to curing.

Figure 5.64 Repair "Bug-Holes" after Placing Shotcrete

Small Volume Pneumatic Concrete

Hand-held machines have been developed for small repairs. These machines are effective in re-pointing laid up stone, re-mortaring granite curbs, and re-mortaring bridge rail post foundations. An example of a small volume shotcrete tool is shown in Figure 5.65.

Figure 5.65 Small Volume Shotcrete Tool

A concrete mix with fine aggregates (mortar sand for instance) is added to a hopper on the tool. Water is added to the dry mix at the nozzle. The success of this application is also dependent on the skill of the applicator. The application, if done properly, provides for a dense concrete with good adhesion. An example of a small volume shotcrete tool application is shown in Figure 5.66.

Some manufacturers offer polymer modified mixes that can be pneumatically applied. These mixes offer the advantages of polymer concretes along with the benefits gained from pneumatically applying concrete to a vertical or overhead surface.

Figure 5.66 Re-Pointing a Laid-Up Stone Parapet With a Small Volume Shotcrete Tool

5.7.7 Finishing

Concrete that will be visible, such as approach slabs, bridge decks, and sidewalks, often needs finishing. Concrete slabs can be finished in many ways depending on the intended service use. Some surfaces may require only strikeoff and screeding (see Figure 5.67) to proper contour and elevation, while for other surfaces a broomed, floated, or troweled finish may be specified (see Figure 5.68). In slab construction, screeding or strikeoff is the process of cutting off excess concrete to bring the top surface of the slab to proper grade. A straight edge is moved across the concrete with a sawing motion and advanced forward a short distance with each movement. A challenge for concrete finishers can be access. Figure 5.69 shows a worker using a bridge to hand finish the concrete.

Figure 5.67 Use of a Screed to Finish Elevation on a Sidewalk

Figure 5.68 Trowel Finish on a Deck Patch

Figure 5.69 Using a Bridge for Concrete Finishing

Bullfloating eliminates high and low spots and embeds large aggregate particles immediately after strikeoff. This looks like a long-handled straight edge pulled across the concrete. Jointing is required to eliminate unsightly random cracks. Contraction joints are made with a hand groover or by inserting strips of plastic, wood, metal, or preformed joint material into the unhardened concrete. Saw cut joints can be made after the concrete is sufficiently hard or strong enough to prevent raveling. After the concrete has been jointed, it should be floated with a wood or metal

hand float or with a finishing machine using float blades. This embeds aggregate particles just beneath the surface; removes slight imperfections, humps, and voids; and compacts the mortar at the surface in preparation for additional finishing operations. Where a smooth, hard, dense surface is desired, floating should be followed by steel troweling. Troweling should not be done on a surface that has not been floated; troweling after only bullfloating is not an adequate finish procedure. A slip-resistant surface can be produced by brooming before the concrete has thoroughly hardened, but it should be sufficiently hard to retain the scoring impression.

5.7.8 Curing

Curing plays an important role on strength development and durability of concrete. Curing takes place immediately after concrete placing and finishing, and involves maintenance of desired moisture and temperature conditions, both at depth and near the surface, for extended periods of time. An example of a wet cure for a small deck patch is shown in Figure 5.70. Pre-soaked burlap covered with polyethylene sheets is used to keep the concrete wet. Properly cured concrete has an adequate amount of moisture for continued hydration and development of strength, volume stability, resistance to freezing and thawing, and abrasion and scaling resistance.

Figure 5.70 Wet Cure of a Small Deck Patch Using Pre-Soaked Burlap

The length of adequate curing time is dependent on the following factors:

- The type of cementitious materials used
- Mixture proportions
- Specified strength
- Size and shape of concrete member
- Ambient weather conditions and
- Future exposure conditions

Slabs on ground (e.g. pavements, sidewalks, parking lots, driveways, floors, canal linings) and structural concrete (e.g. bridge decks, piers, columns, beams, slabs, small footings, cast-in-place walls, retaining walls) require a minimum curing period of seven days for ambient temperatures above 40 °F).

The American Concrete Institute (ACI) Committee 301 recommends a minimum curing period corresponding to concrete attaining 70 percent of the specified compressive strength. The

often specified 7-day curing commonly corresponds to approximately 70 percent of the specified compressive strengths. The

70 percent strength level can be reached sooner when concrete cures at higher temperatures or when certain cement/admixture combinations are used. Similarly, longer time may be needed for different material combinations and/or lower curing temperatures. For this reason, ACI Committee 308 recommends the following minimum curing periods:

- ASTM C 150 Type I cement: 7 days
- ASTM C 150 Type II cement: 10 days
- ASTM C 150 Type III cement: 3 days
- ASTM C 150 Type IV or V cement: 14 days

There are three main functions of curing:

1. To maintain mixing water in concrete during the early hardening process, which is accomplished by:

- *Spraying and fogging:* Spraying and fogging are used when the ambient temperatures are well above freezing and the humidity is low. Fogging can minimize plastic shrinkage cracking until the concrete attains final set.
- *Saturated wet coverings:* Wet coverings saturated with water should be used after concrete has hardened enough to prevent surface damage (see Figure 5.71). They should be kept constantly wet.
- *Left in Place Forms:* Left in place forms usually provide satisfactory protection against moisture loss for formed concrete surfaces. The forms are usually left in place as long as the construction schedule allows. If the forms are made of wood, they should be kept moist, especially during hot, dry weather.

2. To reduce the loss of mixing water from the surface of the concrete:

- *Cover concrete with impervious paper or plastic sheets:* Impervious paper and plastic sheets can be applied on thoroughly wetted concrete. The concrete surface should be hard enough to prevent surface damage from placement activities.
- *Apply membrane-forming curing compounds:* Membrane-forming curing compounds are used to retard or reduce evaporation of moisture from concrete. They can be clear or translucent and white pigmented. White-pigmented compounds are recommended for hot and sunny weather conditions to reflect solar radiation. Curing compounds should be applied immediately after final finishing.

3. Accelerating strength gain using heat and additional moisture:

- *Live steam:* Live steam at atmospheric pressure and high-pressure steam in autoclaves are the two methods of steam curing. Steam temperature for live steam at atmospheric pressure should be kept at about 140 °F or less until the desired concrete strength is achieved.
- *Heating coils:* Heating coils are usually used as embedded elements near the surface of concrete elements. Their purpose is to protect concrete from freezing during cold weather concreting.
- *Electrical heated forms or pads*: Electrical heated forms or pads are primarily used by the precast concrete producers.

- *Concrete blankets:* Concrete insulation blankets are used to cover and insulate concrete surfaces subjected to freezing temperatures during the curing period. The concrete should be hard enough to prevent surface damage when covering with concrete blankets.

Figure 5.71 Preparing for the Cure of the Repair of a Concrete Fascia Beam Repair

Curing in either cold or hot weather requires additional attention. In cold weather, some of the procedures include heated enclosures, evaporation reducers, curing compounds, and insulating blankets. The temperature of fresh concrete shall be above 50 °F. The curing period for cold weather concrete is longer than the standard period due to reduced rate of strength gain. Compressive strength of concrete cured and maintained at 50 °F is expected to gain strength half as quickly as concrete cured at 73 °F. In hot weather, curing and protection are critical due to rapid moisture loss from fresh concrete. The curing actually starts before concrete is placed by wetting substrate surfaces with water. Sunscreens, windscreens, fogging, and evaporation retardants can be used for hot weather concrete placements. Since concrete strength gain in hot weather is faster, curing period may be reduced.

Concrete bridges require a high standard of curing to achieve the low permeability required for protection of steel reinforcement. Standard recommendations for curing bridge decks is moist curing for a minimum of 7 days for concrete mixtures containing only Portland cement and as long as 14 days when supplementary cementing materials are included in the concrete mixture. Some states also require the application of curing compound upon removal of the moist curing methods.

Curing can be problematic for bridge maintenance staff when patching existing concrete decks. Traffic typically needs to be restored in less than 7 days. For this very reason there are specific deck patching products that cure quickly, typically 2 to 4 hours. These patch materials contain special additives or polymers to help in curing. A product popular with many bridge owners allows curing and restored bridge traffic in 45 minutes.

Typical moist curing for bridge decks requires the application of adequate quality water saturated burlap or other approved absorptive material covered with minimum 6 mil plastic covering (see example shown in Figure 5.72). The temperature of the saturated materials should be within 20 °F of the temperature of the in-place concrete. White plastic is used to

reflect solar radiation, reducing the temperature rise beneath the plastic, while cold temperatures (less than 50 °F) may allow the use of black plastic to add heat to the system.

Figure 5.72 Wet Cure with Pre-Soaked Burlap for a Large Deck Repair

5.8 Crack Repair

The cause of cracking should be determined before repairs are undertaken. Structural defects should be remedied before filling. A crack caused by settlement, flexural or diagonal tension will recur if the problem is not corrected.

Cracks can be repaired one at a time or done by a flood coat (also called: topical application). Small cracks occurring over large areas can be sealed with a crack filler applied to the entire surface. These materials have very low viscosity and are able to flow deep into the cracks to seal out moisture. If the cracks are full-depth, it will be necessary to seal off the bottom surface. As these cracks cannot be cleaned out, the bond to the existing concrete may be compromised. The sealant is often an epoxy compound. Urethanes, which remain flexible through large temperature variations, have been used successfully. Rapid setting cementitious mortars may also be used.

Large, open cracks that are dormant or not “working” may be treated by injection. Typically an epoxy or urethane based product is used. The injection method is laborious but is effective in treating vertical cracks. Cracks on horizontal surfaces can be packed with mortar sand, and then a low viscosity polymer is fed into the sand. A lower viscosity product, such as a urethane, should be used for this type of repair.

If cracks are open and appear to be “working”, or if a crack has been sealed but has cracked again, a flexible sealant, such as asphalt crack sealer, should be used. This usually requires routing a groove into the crack to act as a reservoir to hold the proper amount of sealant (see example in Figure 5.73). A backer rod may also be necessary.

Figure 5.73 Modified Squeegee to Contain Low Viscosity Polymer

In all cracking operations, the cause of the cracking should be understood and corrected (if practical). The crack surfaces should be clean and dry and the depth of the seal should be less than or equal to the width of the seal.

The following sections present information on the type, application, and effectiveness of sealers and coatings. Generalized information is provided in Table 5.5 .

Table 5.5 Sealer, Coating and Membrane Properties

Material (Coating/Sealer Type)	Bond	Abrasion Resistance	Chemical Resistance	Elongation	Chloride Ion Resistance	Water Absorption Resistance	Vapor Transmission	Penetration	Ultraviolet Resistance	Life Expectancy	Wear Resistance	Cost
Linseed Oil (Penetrating Sealer)					P	P	P	P	P	L	G	low
High Molecular Weight Methacrylate (HMWM) (Penetrating Sealer)					G	E	F	E	V	E	P	high
Silicate (Penetrating Sealer)					F	P	G	F	F	G	G	low
Siloxane (Penetrating Sealer)					E	E	V	G	G	G	V	mid
Silane (Penetrating Sealer)					V	V	E	E	E	E	E	high
Latex (Surface Sealer 0-30 mils or High Build Coating 10-30 mils)	G	F	P	-								
Silicone (Surface Sealer 0-30 mils)	F	P	P	-								
Methacrylate (Surface Sealer 0-30 mils)	G	G	F	-								
Epoxy (Surface Sealer 0-30 mils or High Build Coating 10-30 mils)	E	G	F	-								
"Hard" Urethane (Surface Sealer 0-30 mils or High Build Coating 10-30 mils)	F	E	G	-								
Elastomeric Urethane (Membrane 30+ mils)	F	F	P	E								
Vinylester (Membrane 30+ mils)	E	G	E	-								
Polyester (Membrane 30+ mils)	G	G	E	-								

Rating Definitions: E – Excellent, V – Very Good, G – Good, F – Fair, L – Low, P – Poor

Additional information can be found in the crack repair guide at:
http://www.virginiadot.org/business/resources/const/GuideManCrackRepair.pdf

5.8.1 Routing and Sealing Cracks

Flexible sealing can repair "working" cracks. The technique consists of: 1) Routing the crack (see example routers in Figure 5.74 and Figure 5.75), 2) Cleaning and drying the crack, and 3) Filling the crack with a suitable field-molded flexible sealant (see Figure 5.76).

Routing and sealing may also repair both narrow and wide cracks that are dormant (non-working) cracks. The method involves enlarging crack along its exposed face and filling and sealing it with a suitable joint sealant (see Figure 5.76). This is the simplest and most common technique for crack repair. It can be executed with relatively untrained labor.

Figure 5.74 Crack Router for Preparing Horizontal Cracks

Figure 5.75 Routing Longitudinal Cracks

Figure 5.76 Placing Crack Sealing into "Working Cracks"

The routing operation consists of preparing a groove at the surface that is sufficiently large to receive the sealant. The groove is prepared by using a concrete saw or pneumatic tools. A minimum surface width of 1/4 inch is desirable. Repairing narrow grooves is difficult. The surface of the routed joint should be cleaned with an air jet and permitted to dry before placing the sealant.

5.8.2 Flooding Application of Crack Sealers

Relatively wide, dormant cracks in bridge decks are effectively repaired through gravity fill polymers such as high-molecular-weight methacrylates, ureathanes, and low viscosity epoxies. To fill these cracks the deck is usually flooded with the polymer. The polymer is brushed into the cracks until they are filled. Usually, sand is spread onto the wet, uncured polymer to provide adequate skid resistance (shown in Figure 5.77). Failure to do so will cause slippery conditions when wet. The material remaining on the deck will abrade under traffic. Though the deck may be unsightly during the process, no harm is done to the concrete.

Figure 5.77 Spreading Mortar Sand onto Topical Crack Sealer

Polymers used to repair cracks by gravity fill have a viscosity of less than 100 centipoise (a measure of viscosity). High-molecular-weight methacrylates that have viscosity of less than 25 centipoise (cp) have been shown to be effective in repairing cracks with widths of 0.008 to 0.08

inch. A minimum crack of 0.02 inch is recommended for gravity fill epoxies that usually have a viscosity of about 100 cp.

5.8.3 Sealing Concrete Cracks by Epoxy Injection

Sealing cracks by a process of injecting epoxy into the crack is very effective in bridge maintenance.

The process of determining the feasibility of epoxy injection of cracks includes:

- Consulting with engineer to determine if feasible. For example, if there are so many cracks that the structural integrity of the element is too compromised for this type of repair.
- Determine the size and depth of the cracks.
- Determine if the cracks are active.

Examples of epoxy injection are presented in Figure 5.78 and Figure 5.79. Before a method is identified for repairing a crack, it must be determined if the crack is working (i.e., active or moving) or not. If the crack is working it should be filled with flexible material. If it is passive a bonding material such as an epoxy can be injected into the crack. It is also necessary to determine if the crack is full depth. This may be difficult if the crack is in an abutment.

Figure 5.78 Epoxy Injection Repair (Horizontal Surface)

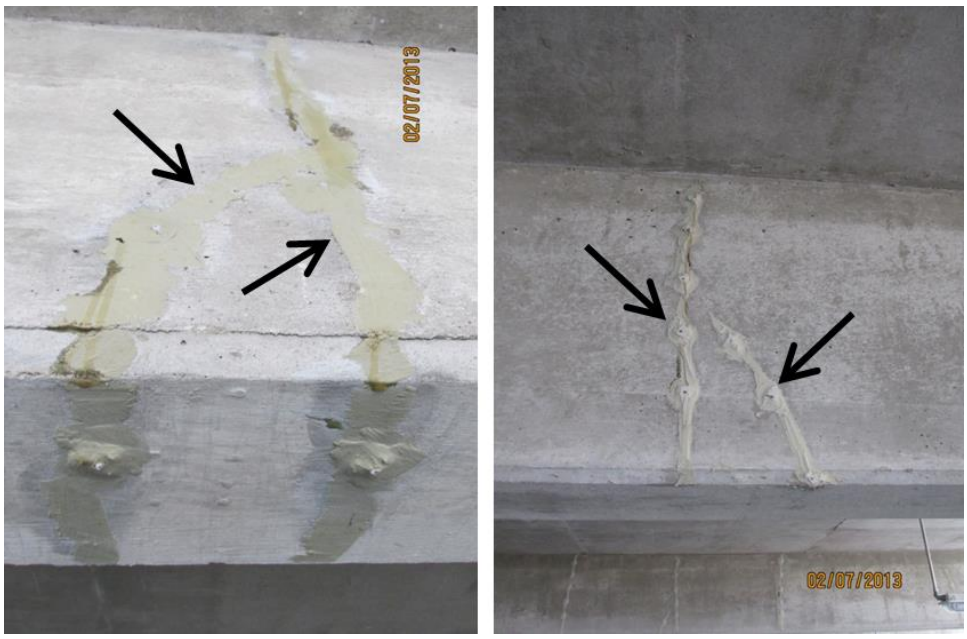


Figure 5.79 Epoxy Injected Cracks (Horizontal and Vertical Surfaces)

Narrow cracks that are dormant (i.e., non-working) may be effectively sealed by epoxy injection. The procedure can be applied to both horizontal and vertical surfaces. Cracks as narrow as 0.002 inch can be sealed and bonded by the epoxy injection. The procedure has potential to provide structural repair (increase stiffness and strength) in addition to sealing the crack.

The technique requires:

1. Cleaning the cracks
2. Sealing of the crack surface
3. Installation of entry and venting ports
4. Preparation of epoxy
5. Injecting the epoxy
6. Removal of surface seal

These procedures are discussed in greater detail below.

1. Cleaning the cracks: Before injection, the interior of the crack should be cleared of all dust, dirt, oil, grease, or fine particles of concrete that could prevent epoxy penetration and bonding. Harsh chemicals or detergents should not be used to clean the cracks because they may compromise the ability of the epoxy to bond to the concrete.

2. Seal the crack surfaces: The exterior of the cracks should be sealed and allowed to harden to prevent the injected epoxy from leaking out of the crack. Cracks can be sealed by applying an epoxy, polyester, or other appropriate sealing material to the surface of the crack. For cracks that extend through the entire member section, the opposite side of the injection should be sealed as well. If the cracks on each side do not connect, epoxy injection should be performed on each side individually. If extremely high injection pressures are needed, the crack can be cut out to a depth of 1/2 inch and width of 3/4 inch in a V-shape, filled with epoxy and struck off flush with the surface.

3. Install the entry and venting ports: There are two general methods that can be used to install the entry and venting ports; surface mounted and socket mounted. Entry ports are typically tube devices that allow the pressurized epoxy resin to be pumped into the crack (see Figure 5.80). The entry port spacing is typically at 8 inches on center but can be increased for wider cracks. Port spacing depends on the crack width and the amount of pressure applied, however the spacing should be limited to the thickness of the repaired member if the cracks pass all the way through. Surface-mounted entry ports are normally adequate for most cracks, but socket-mounted ports are used when cracks are blocked, such as when calcified. In some cases, it may be necessary to drill holes approximately 3/4 inch in diameter and 1/2 to 1 inches below the surface of the crack to place the entry or exhaust port.

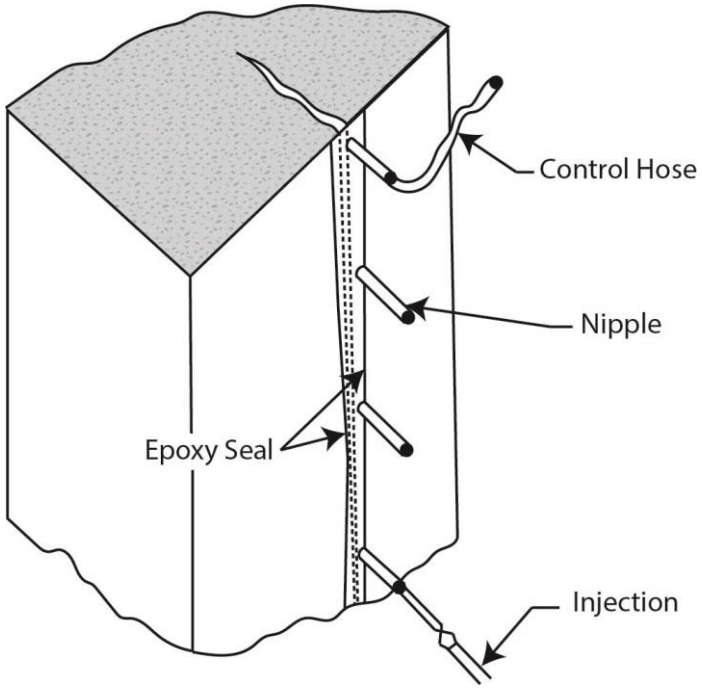


Figure 5.80 Epoxy Injection Schematic (Vertical Surface)

4. Inject Material: The injection progresses from port to port, normally starting at the lowest point, and continuing until epoxy is extruded from the next port. The distance between the ports should not exceed the expected penetration depth. A handgun or pressure pot can be used, but various types of machines are available that assure the proper proportioning, mixing, and temperature of the two part epoxy and the proper injection pressure. This technique may also be used to fill isolated voids or delamination in concrete. In this case, injection pressure must not be too high. This is not, however, considered as permanent as removing and replacing the concrete.

5.8.3.1 Epoxy Repair Safety

Epoxy repairs utilize epoxy resins that are hazardous and appropriate safety precautions should be followed. Safety precautions should include the following:

- Have the Material Safety Data Sheet (MSDS) on site and read it before use
- Wearing protective clothing including rubber gloves and protective eyewear
- Having eye wash kit or facilities available

- Providing ventilation in confined spaces or wear an appropriate air respirators where needed
- Having necessary cleaning materials on hand

Epoxy injection is further addressed in several sections of this document, including:

- 7.2.4.2 Epoxy Crack Injection regarding maintenance of bridge decks and slabs
- 11.4.3 Epoxy Injection Crack Repair regarding maintenance and repair of concrete superstructures
- 13.4.5 Crack Repair regarding repair and rehabilitation of concrete substructures

Additional information can be found in ACI 306.1, *Standard Specification for Cold Weather Concreting*, ACI 306R, *Cold Weather Concreting*, ACI 305.1, *Specification for Hot Weather Concreting*, and ACI 305R, *Hot Weather Concreting*.

5.8.3.2 Quality Control of Epoxy Injection of Cracks

To ensure that the injection has been successful, quality assurance measures may include test cores or NonDestructive Evaluation (NDE).

1. Test cores:

- Core locations should be chosen to avoid cutting reinforcing steel, drilling cores in areas of high stress, or creating core holes below the waterline. The engineer should determine core locations when these types of conditions exist.
- Be sure the epoxy has set before extracting a core.
- Take cores (normally 2 inch diameter) to check that the penetration of the epoxy is adequate.
- Inspect the core visually to determine the penetration of the epoxy into the crack; Cores can be further tested for compressive and split tensile strength per ASTM C 42.
- Subsequently, patch the removed-core area (after proper surface preparation) with an expansive cementitious or epoxy grout compatible with the existing substrate concrete and the surrounding environment.

2. Methods for nondestructive evaluation:

- Impact Echo (IE)
- Ultrasonic Pulse Velocity (UPV)

5.8.3.3 Underwater Epoxy Injection of Cracks

Pressure injection of cracks underwater can be used for underwater applications also. When cracks expose the reinforcing steel to moisture, the stage is set for the corrosion process to begin. In salt water environments corrosion can occur very fast. With the proper selection of a water-compatible adhesive, normally an epoxy resin, dormant (non-moving) cracks saturated with water can be repaired. The procedure can also repair other small voids such as delaminations or honeycombed areas near the surface of the concrete. Pressure injection can be used, within limits, against a hydraulic head provided the injection pressure is adjusted upward to counteract the pressure of the hydraulic head. The material must displace the water as it is injected into the crack to ensure that the crack is properly sealed resulting in a watertight monolithic structural bond.

Epoxies must have certain characteristics to cure and bond the cracked concrete together. Many adverse elements are present inside the concrete crack such as water, contaminants carried by water, dissolved mineral salts, and debris from the rusting reinforcing steel. The typical low surface temperature of concrete underwater eliminates many products due to their inability to properly cure. The epoxy injection resins for cracks are formulated in low viscosity, and they do not shrink appreciably. The surface wettability of epoxy resin is of major importance, because the resin should displace all water in the crack and adhere to a wet surface and then cure in that environment.

The procedure involves cleaning of the crack by a high-pressure water jet system and shaping the surface of the concrete directly above the crack so that it can be sealed with a grout. Injection ports are installed in holes drilled to intersect the crack by a hydraulic or pneumatic drill. Subsequently, the surface of the crack is sealed with a grout material suitable for underwater use such as the cementitious or epoxy mortar described earlier. The purpose of the grout is to retain the adhesive as it is pumped into the crack. The adhesive is pressure-injected into the crack through the ports that are embedded in the grout at regular intervals. The injection sequence begins at the bottom and advances upward. The injection moves up when the adhesive reaches and begins to flow from that port. Epoxy resin is mixed either before or after pumping. Cracks varying in width from 0.002 inch to 0.25 inch may be injected successfully. Epoxy pressure injection has gained widespread acceptance as a cost effective method to bond together and seal cracked structural concrete members.

The following precautions should be noted:

- Contaminants growing inside the crack, especially those found underwater can reduce the successful welding of cracks.
- Corrosion debris can also reduce the effectiveness of pressure injection.
- Time and patience is required for the successful injection project.
- Injection is labor-intensive. (As the temperature drops below 50°F, it becomes more difficult to pump the epoxies into fine cracks).

Experience on the part of the diver in injection and the formulation of the epoxy for injection are very important.

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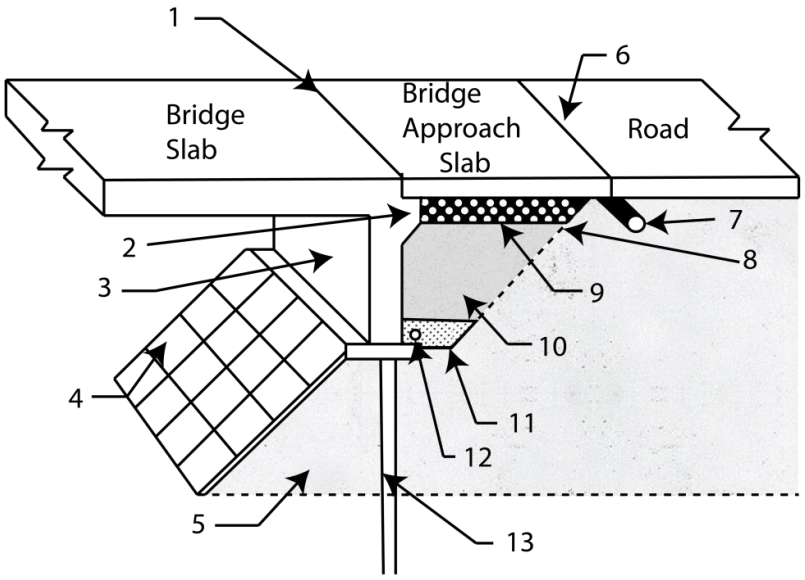
Chapter 6 - Approach Slabs and Approach Roadway

6.1 Approach Slab Function and Potential Issues

What To Look For

- Cracks in Slab
- Uniform/differential settlement of slab
- Degradation/failure of joints with bridge or pavement
- Ponding on slab
- Faulting of slab
- Erosion
- Voids
- "Bumps"

Roadway embankments, which provide support for bridge approach slabs, are continually subjected to traffic-induced compression and settlement throughout the life cycle of the roadway. Embankments are constructed on existing soil subgrades or compacted fill. Figure 6.1 shows the elements of a typical approach slab built on granular backfill.



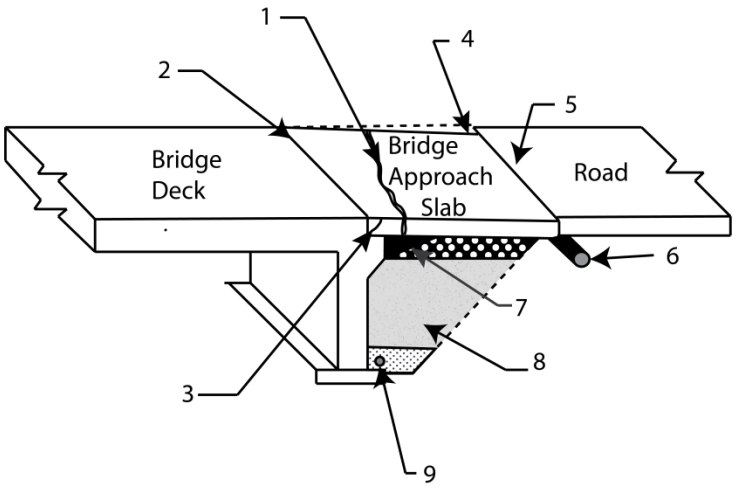
1. Expansion Joint
2. Paving Notch
3. Abutment
4. Slope Protection
5. Bridge Embankment
6. Joint
7. End Drain
8. Backfill Excavation Limit
9. Special Backfill
10. Granular Backfill
11. Porous Fill
12. Subdrain
13. Pile

Figure 6.1 Elements of an Approach Slab

Bridges, by their nature of often having deeper foundations on compact soils, piles or rock, experience less settlement over time. The difference in settlement leads to noticeable bumps, which vehicles feel as they travel across bridges. Approach slabs have often been used to shift the settlement difference from the bridge to further along the roadway, providing a transition zone to minimize noticeable changes in roadway grade.

Some of the potential issues that occur with approach slabs are listed below; some are also shown schematically in Figure 6.2:

- Granular backfills compacting further over time, most often a result of poor compaction during construction (see Figure 6.3)
  - Creates voids
  - Creates faulting of the slab
  - Can lead to cracking of the slab concrete
  - Degradation of joints at both the bridge-slab and the slab-pavement interfaces
- Temperature cycling of deck and approach slabs throughout the years
  - Causes expansion joints to fail
  - Creates voids in winter due to soil compaction during summer bridge expansion
  - Allows water under the slab to erode backfill
  - Longitudinal joints along face of U-wall fail, causing water to erode backfill along edge of slabs
- Short approach slabs (length less than 40 feet)
  - Magnifies settlement issues causing more noticeable bumps
- Approach slabs not built curb-to-curb
  - Leads to differential settlement, especially on sidewalks or at-grade wide shoulders
- Corbels used on abutment stems
  - Prevents proper compaction behind the abutment stem walls
  - Creates pivot point for slabs to rotate on during soil settlement on the other end
  - Leads to expansion joint failure at the back face of the abutment stem
- Cracks appearing through the slab or ponding after precipitation
  - Often indicates settlement of one end, corner, or side of approach slab
  - Usually will continue to grow or spread on the slab if not corrected



1. Faulting
2. Expansion Joint
3. Shearing of End of Approach Slab
4. Settlement of Approach Slab
5. Joint
6. Plugged End Drain
7. Void
8. Poorly Compacted Granular Backfill
9. Plugged Subdrain

Figure 6.2 Potential Issues with Approach Slabs

Figure 6.3 Non-Compacted Granular Backfill Behind a Bridge Abutment (Courtesy of Iowa DOT)

6.2 Assessing Problems

6.2.1 Ride Quality and Settlement Problems

The settlement of the bridge approach pavement has been a continuing problem for many bridge maintenance engineers, leading to bumps and driver discomfort on roadways. Pavement settlement results in poor ride quality on the approach and may pose a safety problem in severe cases of uneven settlement. In cases where bridge approach settlement has created a bump at the end of a bridge, impact loads may be created that can damage the structure. An example of approach settlement is presented in Figure 6.4.

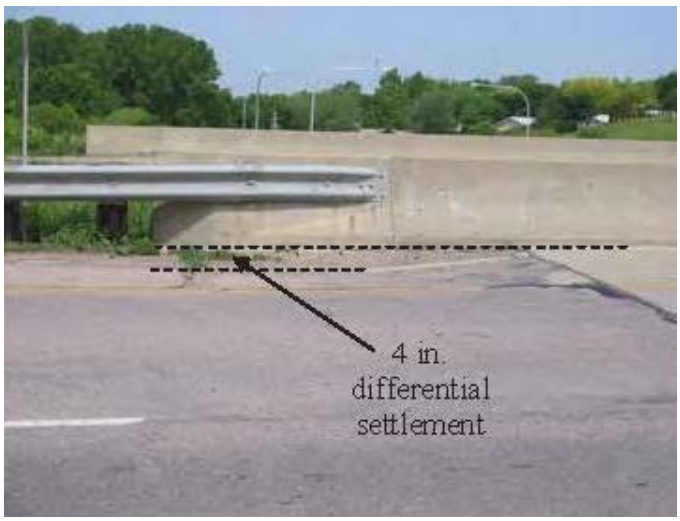


Figure 6.4 Differential Settlement Between the Bridge Deck and the Approach Slab (Courtesy of Iowa DOT)

When concrete approach slabs are attached to the bridge, the settlement is less noticeable. The settlement usually results in cracked pavement that can become severe if it is allowed to continue. Pavement settlement can be a recurring problem. It is often necessary to periodically repeat the repair procedure.

Common causes of fill and approach settlement are provided below. Evidence of these conditions should be investigated when assessing settlement problems at bridge approaches.

- Poorly compacted fill material adjacent to the abutment that continues to consolidate over time.
- Voids behind the abutment resulting from scour caused by the waterway or erosion from surface drainage (see Figure 6.5). Spill through or column abutments are particularly prone to this problem. A small rock drill or ground-penetrating radar can be used to locate and determine the size of voids under the pavement (see Figure 6.6).
- A slight tilting of the abutment from uneven settlement or from scour caused by the waterway that creates a small void near the pavement seat. The pressure of the pavement pushing against the abutment, producing additional movement can further aggravate this situation.

Figure 6.5 Void under Approach Slab (Courtesy of Missouri DOT)

Figure 6.6 Drilling through the Approach Slab (left). Measuring the Depth of the Void (right). (Courtesy of Iowa DOT)

If settlement is occurring, it is very important that the cause of the problem be determined. If scour is creating a void, the condition could become unsafe. Additional layers of blacktop will only temporarily hide the problem and contribute to the unsafe condition because of the increased dead load over the void.

6.2.2 Drainage Problems

Inadequate drainage of abutment backfill is a common problem. Abutment subdrains and weep holes often do not function as intended, or have failed completely, due to being blocked by soil deposits or debris; an example of this is provided in Figure 6.7. Subdrain failure is often a result of non-porous backfill being placed around them. Abutment subdrain failure can lead to erosion of slopes and collapses of armoring.

Figure 6.7 Weepholes Filled with Backfill Material (Courtesy of Iowa DOT)

Surface drainage deficiencies on approach slabs can lead to voids beneath the slabs and eventual settlement. Some items to look out for while performing maintenance activities are provided below.

What To Look For

- Surface gutters and catch basin inlet grates that are clogged by debris
- Sand accumulations along guiderail posts and at inlets or outlets to drain pipes
- Areas where water is or has the ability to pond on top of the approach slab, on the embankments around the abutments, or has previously eroded slopes
- Cracked or damaged expansion joints at the end of the approach slab, where it meets either the deck directly or the rear face of the abutment backwall
- Clogged drainage outlets
- Large separation between bridge railing and return wall

6.3 Preventive and Basic Maintenance of Approach Slabs and Approach Roadway

Proper maintenance of bridge approaches is often as important as proper maintenance of the bridge itself. Problems on the approach may be greater than those of the bridge and can ultimately affect the condition of the bridge, especially in situations where bridge approach settlement has created a bump at the end of a bridge.

6.3.1 Approach Slab Maintenance

To address ride quality issues, maintenance crews may perform the following activities related to approach slabs:

- Patching of spalls (potholes) in the approach slab with asphalt, typically performed on an annual basis
- Placing a temporary asphalt ramp at the end of the bridge when an approach made of poor soils heaves or settles, resulting in a rise or drop of the approach slab in relation to the bridge deck elevation (see Figure 6.8)

Figure 6.8 Asphalt Ramp Placed at the Approach Slab

Preventive maintenance strategies for concrete approach slabs are similar to those employed for concrete bridge decks, and are often performed in conjunction with bridge deck maintenance. Some of the more common preventive maintenance activities are listed below, along with references to the relevant sections in Chapter 7 where these activities are discussed in greater detail.

- Approach slab washing – refer to Sections 7.2.2.1 and 7.2.4.1
- Waterproofing and sealing – refer to Sections 7.2.2.3 and 7.2.4.3
- Overlays – refer to Section 7.2.3, 7.2.4.6, 7.2.4.7 and 7.2.4.8

6.3.2 Maintenance of the Approach Roadway

Maintenance of safety appurtenances in bridge approach areas should also be performed on a regular basis. Such activities include:

- Maintaining all traffic signs
- Prompt replacement of missing or vandalized load limit signs
- Resetting sign posts to a vertical position
- Removing brush, grass, and weeds that obscure signs and reduce visibility
- Repairing deficiencies in guardrails

Other common approach roadway maintenance activities include:

- Grading and shaping of the shoulders adjacent to approach slabs, so that they are at the same (or lower) elevation of the bridge deck
- Removing vegetation under guardrails to facilitate drainage
- Cleaning drainage structures and outletting pipes

6.4 Repair and Rehabilitation of Approach Slabs

6.4.1 Concrete Repair

Typical forms of concrete deterioration in approach slabs, such as cracks, spalls and delamination, are repaired in much the same way as these repairs are performed on concrete deck slabs. Some of the more common concrete repairs are listed below, along with references to the relevant sections in Chapter 7 where these activities are discussed in greater detail.

- Crack injection and epoxy sealing – refer to Sections 7.2.2.6 and 7.2.4.2
- Full and partial depth concrete spall (pothole) repairs – refer to Section 7.2.4.4
- Concrete delamination repair – refer to Section 7.2.4.5

6.4.2 Settlement Repair

There are various methods that can be used to address slab settlement. Installing an asphalt pavement ramp on the approach pavement to compensate for the settlement is a common temporary repair that was mentioned in Section 6.3.1. Alternatively, approach slab overlays, such as those discussed earlier in this chapter, can be applied with varying thicknesses to compensate for the settlement.

Asphalt ramps and approach slab overlays are not appropriate treatments if it is suspected that there are voids beneath the approach slab, as these treatments will simply obscure the underlying problem rather than address it, possibly allowing worsening conditions to go undetected. A few of the methods used by bridge owners to fill voids beneath approach slabs are presented in the following sections.

6.4.2.1 Mudjacking

Mudjacking, also known as Slabjacking, is a process that raises sunken concrete back to its original elevation. The process involves filling any voids and lifting the pavement by pushing it from below with “mud.” The process can cost a fraction of the cost of approach slab replacement, and is relatively fast and clean. The leveling process can be completed and ready for use in a matter of hours. However, if the approach slab contains large areas of deterioration, replacement of the slab may still be a better option.

The mudjacking process requires special equipment and experienced operators. The procedure consists of drilling holes in the pavement and pressure pumping a fine soil slurry containing a hardening agent into the holes. The pressure pumping causes the pavement to rise as the slurry fills any voids as it flows between the pavement and the fill, as shown in Figure 6.9 .

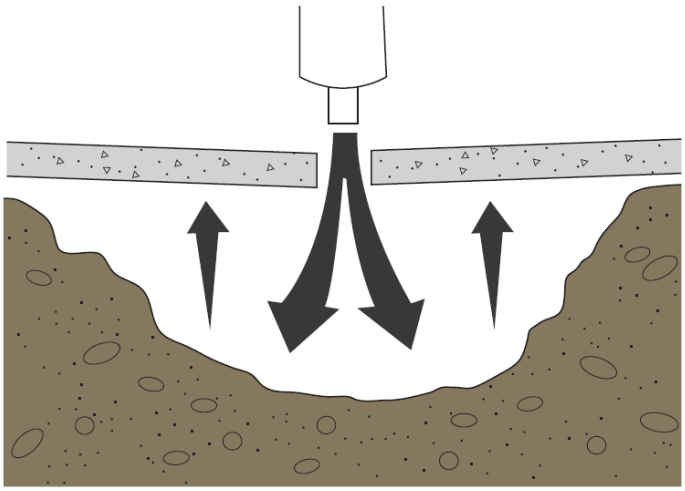


Figure 6.9 Mudjacking Concept

Mudjacking is sometimes also used to fill voids under the approach pavement where settlement has not taken place. The procedures to be followed are basically the same except that the pressure on the slurry should be held to a minimum because lifting of the slab is not required and the probability of blowout is reduced.

There are many materials that can be pumped. Some research is required on the part of the user to find the appropriate material or combination of materials. Some of the most common materials are cement, agricultural lime, pond sand, fly ash, or combinations of these materials. Sufficient water is added to make slurry workable. The characteristics that the material should have are that it is inexpensive, readily available, and is a material that will pump under pressure without having the carrier water separate from the material.

Coarser materials or sand allow separation to happen and can cause many problems. Usually when this separation occurs, it is easily corrected by adding a lubricating material to the mix. The proportion of cement can vary from 1 percent to 20 percent depending on the application or the base material being used. Also, for a mudjacking operation to be successful, it is important that the mix does not shrink as it hardens. Table 6.1 lists the components of typical mixes that have proved satisfactory for mudjacking operations.

Table 6.1 Mudjacking Mixes

	Mix 1	Mix 2	Mix 3
Components	6 parts sand 2 parts commercial (hydrated) lime 1 part Portland Cement*	4 parts sand 8 parts sugarbeet lime (or 12 parts marl) 1 part Portland Cement* Cut-back asphalt RC-1, or asphalt emulsion AE-2, or AE-1 M at a rate of 6.2 liters/cubic meter (1.5 gallons per cubic yard) of slurry	2 parts sand 4 parts commercial (hydrated) lime 1 part Portland Cement* Cut-back asphalt RC-1 , or asphalt emulsion AE-2, or AE-1 M at a rate of 6.2 liters/cubic meter (1.5 gallons per cubic yard) of slurry

* In place of Portland cement, an expansive type cement can be substituted to counteract mix shrinkage.

The fine sand used may be any type having well-rounded particles. Sufficient water must be added to each of the above mixes to make the slurry workable. Mix 2 may also be used without the asphalt materials.

Suggested Procedure

Mudjacking

1. A grid pattern of holes are drilled through the pavement. The diameter of the hole should fit the nozzle on the mudpump. Spacing of the holes depends on the condition of the concrete pavement and the amount of lifting to be accomplished. One typical pattern consists of two rows of holes, each 3 feet from the edge of a 12 foot traffic lane with the holes 6 feet apart along the lane and equally staggered from the holes in the other row. When adjacent lanes are drilled, the holes are staggered from those in the nearest row of the other lane.
2. Once drilled, flush all holes with water under pressure to develop a void under the slab for the slurry to enter.
3. Seal all except two adjacent holes with wooden plugs or stakes.
4. Pump the slurry into one of the two open holes, awaiting the slurry to appear at the other hole, as shown in Figure 6.10.
5. Cover one of the current open holes and remove the seal from a different hole.
6. Fill the new hole with the slurry until it is escaping from the now second open hole and repeat the process until all holes have been pumped into and the approach slab has been raised to the intended elevations. Be sure not to place too much in one hole as that has led to additional cracking of the approach slabs. Smaller placements of slurry will result in better lifting and less slab cracking.
7. Once mudjacking is complete, fill all holes with high-early strength concrete.

An example of mudjacking slurry being pumped under an approach slab is shown in Figure 6.10.

Figure 6.10 Mudjacking Slurry Pumped Under Approach Slab (Courtesy of Missouri DOT)

If slurry appears through the shoulder, as shown in Figure 6.11, an old grader blade can be wedged along and below the pavement edge. Shoulder material can then be added and compacted to seal off the blowout. A wide plank, with the dual wheels of the mudjacking equipment placed upon it, is also effective in keeping the shoulder material from blowing out.

Figure 6.11 Mudjacking Slurry Blowing Out at Barrier (Courtesy of Missouri DOT)

Where the settled pavement areas to be raised are adjacent to structures, extreme caution should be exercised to prevent the concrete pavement approach slab from rising above the bridge deck. The following method has proven effective in preventing over-raising the slab:

Suggested Procedure

Preventing Over-Raising of Slab During Mudjacking

1. Drill 7/8-inch holes diagonally through the approach slab and into the bridge deck slab.
2. Insert 3/4-inch "L" shaped pins into the holes to pin down the approach slab and prevent over-raising of the bridge deck joint. Usually only two or three pins are needed on a 24-foot pavement.
3. After raising the approach slab, remove the "L" shaped pins and refill the holes with mortar.

6.4.2.2 Injecting Expansive Polyurethane

A contractor has developed a method that involves injecting expansive polyurethane under the approach slab to lift the pavement back to its original profile; this method is depicted in Figure 6.12. Grout holes are drilled through the existing approach slab, and then injected by a pumping truck with expansive polyurethane foam that, as it expands, will lift the approach slab and provide an underseal to the slab. Within 15 minutes, 90 percent of its maximum compressive strength is reached. The holes are then filled with non-expansion grout to prevent water and debris from infiltrating the slab. Tests in Oklahoma, Michigan, and Texas found this method to be effective for at least 10 years when properly conducted.

Figure 6.12 Injecting Expansive Polyurethane under Approach Slab (Courtesy of Iowa DOT)

Tests have shown that, prior to injection of expansive polyurethane, the size of the voids should be determined using techniques such as ground-penetrating radar, so that the estimate of expansive polyurethane is more accurate. Fine cracks were also noticed in the slabs, similar to those shown in Figure 6.13, and were likely induced by stresses formed during the slab lifting process.

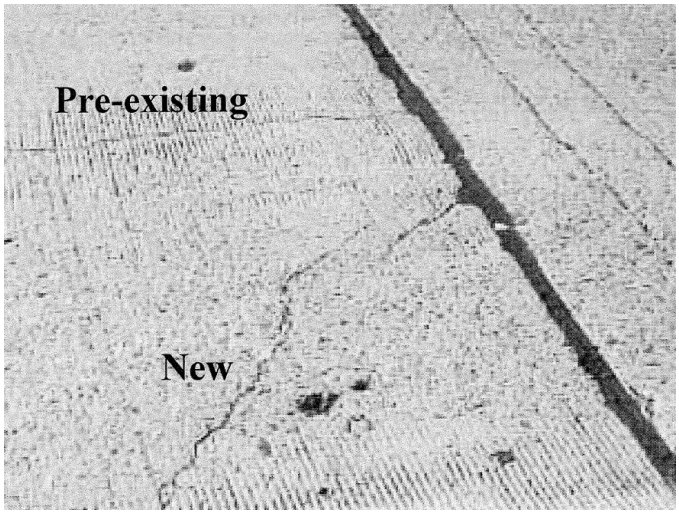


Figure 6.13 Surface Cracks Following Injection of Expansive Polyurethane (Courtesy of Oregon DOT)

6.4.2.3 Pressure Grouting under Slab

Pressure grouting to fill in the voids within the first year of new approach slabs or approach pavement reconstruction has been shown to be effective in settlement prevention. Grouting is done twice, once within the first 2-to-6 months, and then again within 6 months of the first grouting, to fill voids created by settlement and temperature-induced expansion. Grout mix often used was one part Type 1 Portland Cement and three parts Class C fly ash, with water added for workability and fluidity for pumping into holes drilled through the existing slab in-place. However, while used as a mitigation method, this solution is more a short-term fix than a long-term method as soils will continue to settle below the grouted zone.

6.4.2.4 Compaction or High-Pressure Grouting under Slab

High-Pressure Grouting under the slab involves utilizing and often increasing the voids by compacting the soil and fill layers beneath by a sand-cement grout that is placed under high pressures. Once the soil is fully compacted, additional grouting can be used to lift approach slabs and abutting roadways. This method uses a grid pattern of holes drilled through the existing approach slab, and works well with gravel, sand, or coarse silt subgrades.

6.5 Approach Pavement Expansion and Its Effect on Structures

Considerable damage to bridge components has been caused by pressure from encroaching pavements due to the expansion of concrete roadways, also known as "pavement growth." This expensive and irreversible damage may be reduced if bridge engineers, inspectors, and maintenance workers identify the signs of pavement growth early, and initiate corrective action through the installation of a pressure relief joint in a timely manner.

There are common signs that pavement growth may be occurring at a structure. Several examples are provided below.

What To Look For

- Closed expansion joints (see Figure 6.14)
- Damaged or crushed concrete barriers (see Figure 6.15)
- Railing tube separation at joints between two sections of steel or aluminum railing (see Figure 6.16)
- Diagonal deck cracking at ends of bridge (see Figure 6.17)
- Closed pin and hangers, especially if during cold weather
- Severely tilted rocker bearings, especially if during cold weather
- Beam end contact with backwall, or a backwall that is not vertically plumb

Figure 6.14 Expansion Joint Permanently Closed due to Approach Pavement Growth (Courtesy of MDOT)

Figure 6.15 Crushed Barrier due to Approach Pavement Growth (Courtesy of MDOT)

Figure 6.16 Aluminum Tube Rail Separation, a Possible Sign of Approach Pavement Growth (Courtesy of MDOT)

Figure 6.17 Diagonal Deck Cracking May be a Sign of Approach Pavement Growth (Courtesy of MDOT)

6.5.1 Sources of Pavement Growth

There are many contributing factors that can lead to pavement growth, and also determine the rate and extent of pavement growth. These factors include:

- Length of concrete pavement and joint spacing adjacent to the bridge
- Pavement age
- Average daily traffic
- Joint seal maintenance
- Temperature fluctuations
- Use of sand during winter maintenance operations

Often these factors are interconnected and work to complement one another. The average daily traffic a roadway experiences and joint seal maintenance are one such example. It is often difficult to justify lane closures for joint sealing in a metropolitan area, but joint seals wear and deteriorate faster where there is more traffic. Open contraction joints allow debris buildup and thus reduce the amount pavement can expand during increasing temperatures.

In various combinations these sources of pavement growth can cause pressures in excess of 1,000 psi at bridges. Joint sealant maintenance and short longitudinal spacing between contraction joints extends the amount of time before bridge damage occurs. Understanding the pressure generation cycles, as illustrated Figure 6.18, and the process used to alleviate these pressures, will reduce the need for expensive rehabilitation projects.

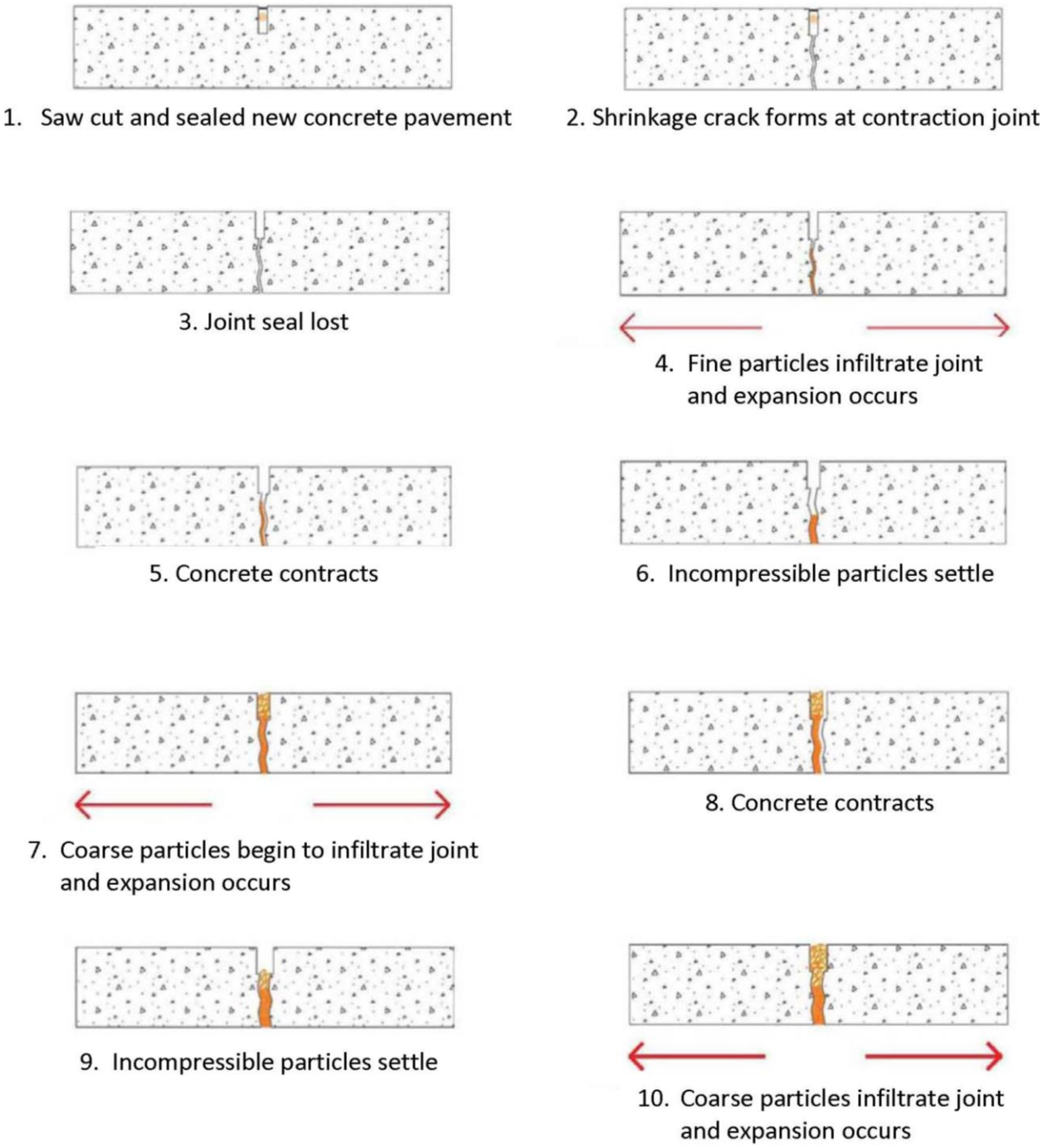

1. Saw cut and sealed new concrete pavement
2. Shrinkage crack forms at contraction joint
3. Joint seal lost
4. Fine particles infiltrate joint and expansion occurs
5. Concrete contracts
6. Incompressible particles settle
7. Coarse particles begin to infiltrate joint and expansion occurs
8. Concrete contracts
9. Incompressible particles settle
10. Coarse particles infiltrate joint and expansion occurs

Figure 6.18 Concrete Pavement Pressure Generation Cycles (Courtesy of MDOT)

6.5.2 Installing Pressure Relief Joints

Pressure relief joints should be located approximately 50 feet away from the structure, but not placed at an existing joint or load transfer device. When installing pressure relief joints, a section of the concrete pavement is saw cut and then removed across the entire width of the roadway. After saw cutting, the concrete and any mesh reinforcement should be removed completely and the aggregate base must be visible. This may be accomplished using either 60 pound jackhammers or a skid steer with mounted hydraulic hammer attachment.

All additional sources of pressure also need to be relieved; these sources could be located within shoulders, curbs, sidewalks and barriers. When cutting concrete barriers, be mindful of the possible presence of electrical conduits, especially if light standards are mounted on the barrier.

6.5.2.1 Bituminous Pressure Relief Joints

For bituminous pressure relief joints, the opening is normally 12 inches wide (minimum) and filled with bituminous material, like the example shown in Figure 6.19.

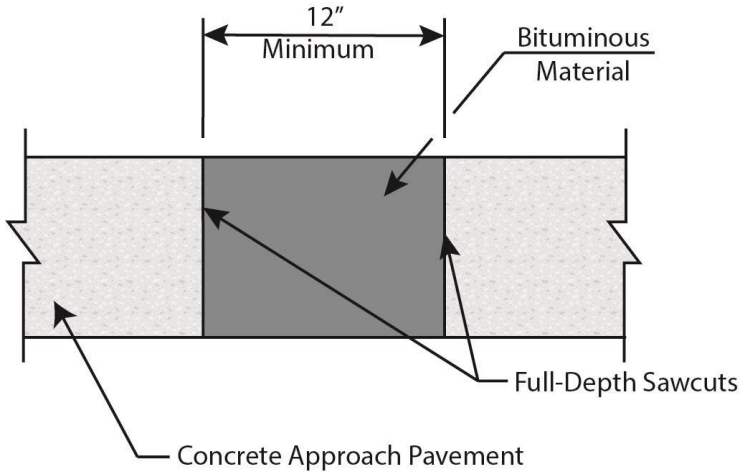


Figure 6.19 Example Detail of a Bituminous Pressure Relief Joint

Bituminous pavement relief joints may bump up over time as pavement growth continues, requiring periodic milling to maintain satisfactory ride quality.

6.5.2.2 Closed Cell Joint Seal Pressure Relief Joints

Narrower pressure relief joints, say 4 inches, can be filled with bridge deck joint seals, as illustrated in Figure 6.20. The joint seal material used should be tested and capable of compressing down to 1 inch without failing or popping out. For this type of pressure relief joint, it is extremely important that the two saw cuts are parallel. Also, installations should only be scheduled prior to peak seasonal temperatures, as placing the material during the summer or fall often results in loss of the material.

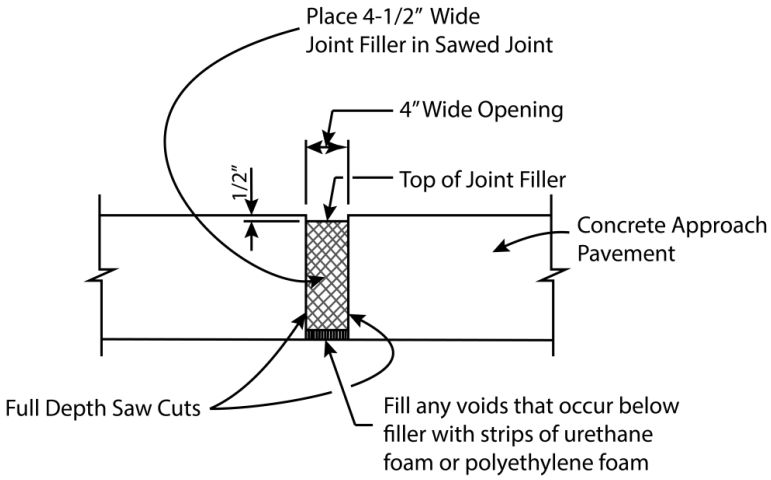


Figure 6.20 Example Detail of a Closed Cell Joint Seal Pressure Relief Joint

Suggested Procedure

Closed Cell Joint Seal Pressure Relief Joint Installation

1. Make two parallel full-depth saw cuts to create a 4-inch-wide joint opening. The joint must be continuous extending across the entire road, and any adjoined curb and gutter or concrete barrier wall must be cut as well.
2. After saw cutting, completely remove the concrete and any mesh reinforcement until the underlying aggregate base is visible. This may be accomplished using either 60 pound jackhammers or a skid steer with mounted hydraulic hammer attachment. A vacuum truck is helpful to remove slurry located in the bottom of the joint that is generated during saw cutting.
3. After concrete removal, dry the vertical surfaces with air and sandblast the vertical surfaces to aid the bonding ability of the lubricant adhesive (see Figure 6.21).
4. Coat each contacting edge of closed cell relief joint material with lubricant adhesive. Inexpensive disposable rollers or steel trowels may be used to spread the material conservatively.
5. Once the sides are coated, press the material in place by hand working from one end to the other without stretching it. Press the foam into the void so it is recessed approximately one half inch. Tools may be cleaned with xylol or acetone to remove any lubricant adhesive residue.
6. The use of spacers is required in areas where irregular saw cuts have resulted in areas where the opening is wider than 4 inches. Spacers are simply pieces of the same material and act as wedges to hold the material in place until it is in a state of compression. Provide spacers a maximum of every 6 inches or as needed to maintain the joint elevation.
7. Allow any excess lubricant adhesive lying on the road surface to cure or place blasting sand on it to prevent tracking. Do not reopen lanes to traffic if the material will be picked up by vehicular traffic.

Figure 6.21 Sandblasting the Vertical Surfaces of the Joint Opening (Courtesy of MDOT)

6.5.3 Monitoring Pressure Relief Joints

Pressure relief joints should be measured regularly and additional joints constructed as the pavement continues to grow. If a joint measures less than 2 inches, a replacement should be added to the list of annual maintenance recommendations. If a pressure relief joint measuring less than 1 inch is discovered, it should be replaced immediately to eliminate the opportunity for damage.

The amount of time required before performing a replacement will vary for each pressure relief joint. A concrete repair project or bituminous overlay on the concrete roadway will significantly reduce the rate at which pavement stresses form. However, if no changes occur and the factors that cause pavement growth are plentiful, replacements will often occur after only four years of service.

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Chapter 7 - Maintenance and Preservation Techniques for Bridge Decks and Slabs

7.1 Bridge Decks and Slabs Maintenance Introduction

The deck or slab of a bridge is like the roof on a house; it receives the brunt of the storm and shelters the contents below from the harsh elements. The deck is vulnerable to the effects of mechanical wear from traffic induced deflections, tire wear, and from environmental conditions such as rain, snow and ice. Consequently, decks and slabs typically require more maintenance and repair than any other bridge component. The following sections in this chapter will present common maintenance and preservation techniques for concrete, steel, timber and fiber reinforced polymer decks.

7.2 Concrete Decks

7.2.1 Introduction

The most common bridge deck and slab material is concrete. The physical properties of concrete permit placement in various shapes and sizes, providing the bridge designer and the bridge builder with a variety of construction methods. Concrete bridge decks resist wear and weather well, provide good traction and ride quality, and in many designs provide a part of the structural strength of the bridge. Careful design, proper selection of materials, and good construction practices are essential for durable bridge decks and slabs.

Among the design features that improve durability are adequate cover over the reinforcement, good drainage, smaller bridge skews, and thicker slabs. Material and construction practices that contribute to durability include achievement of the specified cover, use of air-entrained concrete in areas subject to freezing, use of concrete with appropriate water-cement ratios, good consolidation, and proper curing procedures that prevent excessive shrinkage cracking.

The main cause of concrete deck and slab deterioration is corrosion of the reinforcing steel within the deck. Corrosion is typically caused by small cracks in the concrete that provide pathways for water and chlorides to penetrate the concrete and attack the steel (see Figure 7.1). Once the steel begins to corrode, the steel corrosion by products expand and cause deck delaminations or spalling. Deck corrosion is particularly problematic in states that use deicing salts to melt snow in the winter. To slow the corrosion cycle, the water and chlorides must be kept from reaching the reinforcing steel.

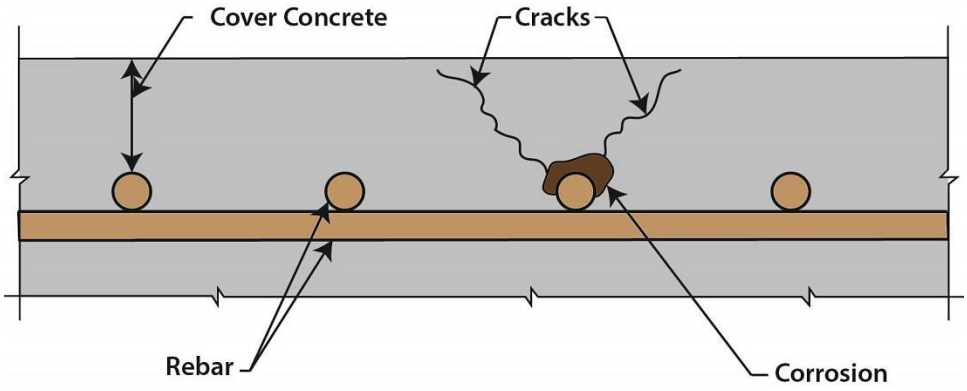

Figure 7.1 Typical Reinforced Concrete Deck and Slab Deterioration

Sealing the bridge deck cracks and overlaying the surface with dense materials will slow the rate of deterioration by limiting the water and salts that reach the reinforcing steel. If a significant amount of chlorides have already reached the reinforcing steel, cathodic protection may be used to stop corrosion from progressing. This section presents various aspects of concrete bridge decks and slabs and the related bridge maintenance activities.

What To Look For

- Cracks
- Spalls
- Leaks on underside of deck
- Efflorescence in edges or underside of deck
- Accumulated debris on deck
- Evidence of ponding on deck

7.2.2 Deck Protection Methods

Proactive deck protection is the best way to preserve the life of any concrete bridge deck or slab. Deck protection systems include a variety of methods that prevent water and chlorides from reaching the reinforcing steel in bridge decks to slow or stop corrosion of the steel.

- Washing and cleaning
- Corrosion resistant reinforcing steel
- Water proofing methods
- Concrete crack control
- Cathodic/Galvanic protection
- Crack sealers and fillers
- Overlays

7.2.2.1 Washing and Cleaning

Bridge washing is a proactive corrosion protection method that utilizes water applied to the bridge deck to flood the deck and rinse away or dilute the accumulation of dirt and chlorides. By flushing the deck with water, the concentration of chlorides in the concrete surfaces is

reduced and the potential for corrosion from the salts is minimized. Procedures for washing a bridge deck are provided in Section 7.2.4.1.

7.2.2.2 Corrosion Resistant Reinforcing

One way to protect reinforcing steel against corrosion is to coat the bars with epoxy. The epoxy prevents the water and chlorides from reaching the steel and thereby minimizes the corrosion. Epoxy coated reinforcing steel can also be used in bridge deck and slab repairs in areas where deicing salts are routinely applied. Epoxy coated bars can be identified by their bright colors, typically green or purple. An example of epoxy coated reinforcement is shown in Figure 7.2. Epoxy coated bars have been shown to delay the onset of corrosion when installed properly.

Figure 7.2 Epoxy Coated Reinforcement

Epoxy coated bars can be nicked or cut very easily during delivery, handling, placement and pouring of concrete. These nicks and cuts can minimize the effectiveness of the epoxy coating. When working on decks with epoxy coated bars, any nicks, cuts or breaks in the coating surface should be repaired using brush applied epoxy before placing new concrete. Epoxy coated bars also require longer lap lengths in order to develop the full strength of the bars.

Stainless steel clad and solid stainless steel reinforcing bars have also been used in bridge decks to protect against corrosion. Stainless steel bars are very expensive but will greatly minimize the corrosion concern in the deck. An example of stainless steel reinforcing bars is shown in Figure 7.3.

Figure 7.3 Stainless Steel Reinforcing (Courtesy of FHWA)

Galvanized reinforcing bars can also be used to protect against corrosion and extend the useful life of bridge decks and slabs. Galvanized bars are hot dipped into a bath of zinc that fuses to the steel surface creating a sacrificial coating that prevents corrosion. An example of galvanized bars is shown in Figure 7.4. Galvanized bars stand up well during construction without any special care.

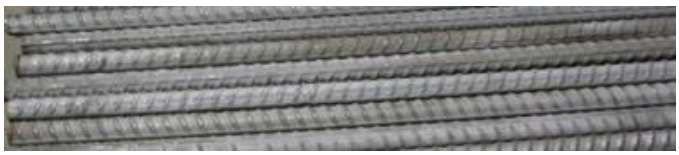

Figure 7.4 Galvanized Bars

Fiber reinforced polymer (FRP) reinforcing steel is another option for a corrosion free bridge deck. FRP is not steel at all and therefore does not corrode. An example of FRP reinforcing steel is shown in Figure 7.5. FRP is well suited for bridge deck applications because it is light and strong and completely corrosion free. The drawbacks to FRP are the initial costs and field workability.

Figure 7.5 FRP Deck Bars (Courtesy of Hughes Bros)

Protection of reinforcing steel may include many types of epoxy or galvanized coatings, the use of stainless steel rebar, or FRP bars.

Other corrosion resistant reinforcing is available under trade names such as MMFX, Z-Bar and special formulations of steel designed to resist corrosion such as low carbon or chromium steel bars. These corrosion reinforcing bars are generally more expensive than conventional rebar but can provide additional corrosion protection. The total life cycle cost of the deck should be considered when considering the additional up front cost of specialty reinforcing bar types.

Any of these methods will improve the corrosion resistance over conventional black bar reinforcing. Reinforcing bar types are compared in Table 7.1:

Table 7.1 Reinforcing Bar Comparison

Reinforcing Bar Type	Corrosion Resistance	Field Workability
Black bars	Fair	Excellent
Epoxy Coated	Good	Good
Galvanized	Good	Good
MMFX, Z-Bar etc.	Excellent	Good
FRP bars	Excellent	Fair
Stainless Steel	Excellent	Excellent

7.2.2.3 Waterproofing and Concrete Sealers

Reinforced concrete bridge decks and slabs can be protected from the penetration of water and chlorides that can lead to corrosion through the application of waterproofing treatments. Common waterproofing treatments include: Epoxies, Silanes and Siloxanes. The sealers can be divided into two general classes: penetrating sealers and film formers. Silanes, siloxanes, and High Molecular Weight Methacrylate (HMWM) penetrate surface cracks and voids in the concrete and limit the penetration of water and salts without leaving a film on the surface. Epoxy healer sealer, methacrylate, and polyurethanes produce a thin film that covers the surface area of the treated deck or slab to protect the concrete from penetrating water and salts. Thin epoxy overlay both penetrates and is a film former. These treatments are typically applied to reinforced concrete decks and slabs that are new or in good overall condition as a preventive treatment. A summary of these treatments is shown in Table 7.2 below.

Table 7.2 Waterproofing and Sealer Summary

Sealer Type	Type	Thickness
Silane	Penetrating	0
Siloxane	Penetrating	0
High Molecular Weight Methacrylate (HMWM)	Penetrating	0
Epoxy Healer Sealer	Penetrating/Film Former	3 - 10 Mils
Thin Epoxy Overlay	Film Former	250 – 375 Mils
Methacrylate	Penetrating/Film Former	20 - 30 Mils
Polyurethanes	Film Former	3 - 5 Mils

7.2.2.4 Concrete Crack Control

Crack control is another proactive deck preservation measure that is carried out during the placement and curing of a concrete deck or slab. Minimizing the extent of concrete cracking

from the initial construction and curing will slow down the ingress of water and chlorides and may eliminate the need for future crack sealing. Crack control can be improved through proper concrete mix design, appropriate water cement ratios, quality placement techniques and proper curing.

The temperature of concrete during the curing process can be a significant factor in control of early age deck cracking. The temperature of the curing deck can be controlled by scheduling deck pours during cooler months or times of the day. Proper wet curing techniques and the application of curing compounds can also minimize the rate of water evaporation from the concrete surface during the curing process resulting in less early age deck cracking. Deck and slab cracking can also be minimized through the use of shrinkage control ad mixtures or through the use of fibers mixed into the concrete. The inclusion of fibers in the concrete mix, an example of which is shown in Figure 7.6, has been shown to significantly control early age deck cracking.

Figure 7.6 Concrete Fiber Reinforcement

7.2.2.5 Cathodic and Galvanic Protection

Corrosion protection can also be supplied through the use of cathodic or galvanic protection systems. Galvanic systems utilize sacrificial anodes that will preferentially corrode instead of the deck reinforcing steel. Cathodic protection systems typically involve electrical currents that are imparted on the reinforcing steel or conductive materials applied to the deck effectively breaking the corrosion process. Various attributes of corrosion protection systems for reinforced concrete bridge decks are summarized in Table 7.3:

Table 7.3 Corrosion Protection Methods

Goal	Method	Expected Annual Maintenance	Expected Life	When to Apply
Corrosion Prevention	Sacrificial Anodes	Minimal	10 – 20 years	New concrete
Corrosion Control (After corrosion has initiated)	Sacrificial Anodes	Minimal	10 – 20 years	No corrosion damage in the concrete
Corrosion Protection	Impressed Current	Moderate	Life of Deck	Stop ongoing corrosion

7.2.2.6 Crack Sealing

Cracks in concrete bridge decks and slabs can be sealed using commercially available penetrating sealers appropriate for widespread cracking, as shown in Figure 7.7, or through pressure injection for larger discrete cracking, as shown in Figure 7.8. Sealing deck and slab cracks will reduce the water and chlorides that are able to reach the reinforcing layer and cause corrosion.

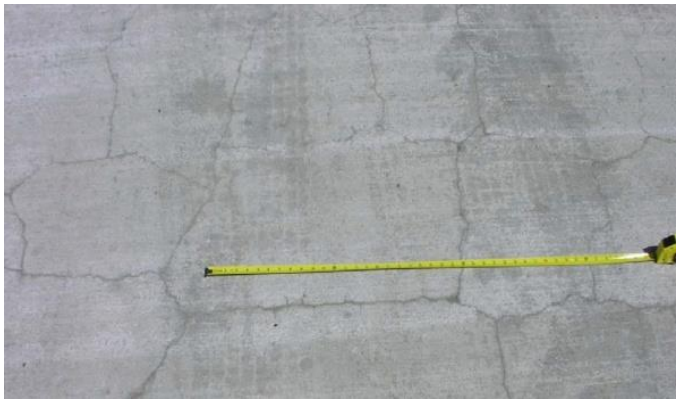

Figure 7.7 Typical Widespread Cracking Pattern on a Bridge Deck (Courtesy of Virginia DOT)

Figure 7.8 Typical Discrete Cracking Pattern on a Deck (Courtesy of Virginia DOT)

Depending on the nature of the cracking observed, a number of methods are available to waterproof, seal or repair the cracks.

Penetrating Sealers

Bridge decks and slabs that have cracks over a large portion of the surface area are good candidates for penetrating sealers. Penetrating sealers are applied to bridge decks and slabs by spraying or pouring on the deck and spreading over the entire area using brooms or squeegees. Penetrating sealers are typically low viscosity which allows them to be drawn into concrete cracks through capillary action. Viscosity is measured by the force per unit area resisting a flow and is measured in the units of centipose.

For comparison, several viscosities are presented:

Water ("low" viscosity): Around 1 centipose

Methacrylates: Around 10 centipose

Epoxy based healer sealers: Around 80 centipose

Epoxy based thin epoxy overlays: Typically 1500 centipose (similar to honey or heavy weight motor oil)

The sealers fill the concrete cracks and voids. On curing, the sealers forms bonds to the walls of the cracks that effectively seal the cracks. The crack sealers significantly reduce the permeability of the concrete. The permeability reduction minimizes the ingress of water and chlorides from road salts or deicing chemicals. There are many commercially available penetrating cracks sealers that are designed for specific applications. The selection of the appropriate crack sealer depends on a number of variables including: crack width, available cure times and desired flexibility of the cured product.

Crack Injection and Epoxy Sealing

Concrete deck and slab cracks that are discrete or localized can be sealed or injected with epoxy resins or other polymers to fill the void and re-bond the concrete surfaces back together. Epoxy injection can restore structural integrity and reduce moisture penetration through concrete cracks 0.002 inches in width and greater. (ACI RAP Bulletin 1: *Structural Crack Repair by Epoxy Injection*).

Full-depth deck cracks must be capped from both the top and below before they are pressure injected from top. Caps have been successfully installed using epoxies, polyesters, paraffin wax, and silicone caulk. The selection of the cap material should consider the following criteria, subject to the type of crack to be repaired: Non-sag consistency (for overhead); Moisture-tolerance; Working life; and Rigidity (modulus of elasticity). (ACI RAP Bulletin 1: *Structural Crack Repair by Epoxy Injection*).

For cracks that are opening and closing under traffic loads, a flexible elastomeric crack sealer may be warranted. The following conditions may indicate a working crack:

- The crack is open and shows evidence of movement
- Has been sealed before and has failed again
- Concrete dust fills the crack

The injected crack sealer material should be designed for the anticipated movement of the deck or slab crack.

A summary of the options for bridge deck crack sealing are presented in Table 7.4:

Table 7.4 Crack Sealer Summary

Attribute	Silane/Siloxane	Heavy Weight Methyl Methacrylate, Polyurethanes	Epoxy Healer Sealer	Thin Epoxy Overlay	Injection Epoxy
Typical Applications*	Waterproofing good condition concrete	Crack sealers for widespread fine cracking	Crack sealers for widespread fine cracking	Crack sealer for widespread discrete larger crack widths	Crack sealer for widespread discrete larger crack widths
Viscosity (Centipoise)	<1	20 - 200	50 - 150	1500 - 3000	500 - 2000
Pot Life (Minutes)	NA	5 - 45	20 - 60	15 - 30	15 - 30
Minimum Cure Time (Hours)	1 - 4	2 - 12	3	1	2
Skid Resistance	Original Surface	Broadcast fine aggregate (sand)	Broadcast fine aggregate (sand)	Broadcast fine aggregate (sand)	Broadcast fine aggregate (sand)
Expected Useful Life (Years)	3 - 5	5 - 10	5 - 10	10 - 15	5 - 10

*Sealers should be formulated specifically for the crack(s) being addressed.

7.2.3 Deck and Slab Overlays

Bridge deck and slab overlays provide a wearing surface that can protect the underlying deck and provide a maintainable layer on the riding surface. Wearing surfaces add protection for the underlying deck or slab but also add dead load weight to the bridge and thus reduce the live load capacity. Adding a new wearing surface or increasing the thickness of an existing wearing surface should be evaluated by an engineer for impact on the live load capacity. Wearing surfaces can generally be categorized into three general classes; asphalt, concrete, and polymer overlays.

When to Call the Engineer

When deciding if an overlay should be applied to an existing deck, an engineer should be consulted to assure that the bridge can handle the extra load.

7.2.3.1 Asphalt Concrete Overlays with Waterproofing Membrane

Asphalt concrete overlays are very common because they are easy to install, relatively inexpensive and most agencies have established mix specifications and a long history of using the product. Asphalt overlays can vary in their performance depending on the density, aggregate and binders used. Asphalt does tend to trap and hold moisture and chlorides that can lead to deterioration of the underlying deck if not properly protected. It is a good practice to include a waterproofing membrane under an asphalt overlay unless the asphalt is a temporary measure to improve ride quality on bridges or bridge decks planned for rehabilitation or replacement.

Water proofing membranes provide a moisture barrier between the asphalt and the underlying deck material. Membranes can be categorized as constructed-in-place or preformed membrane types.

Constructed-in-place membranes include:

- Bituminous
- Spray applied resins

Preformed membranes include:

- Asphalt-impregnated fabrics
- Polymer
- Elastomeric
- Asphalt-laminated board systems

The most commonly used water proofing membranes are bituminous and asphalt-impregnated fabrics. Both common membrane systems consist of a tack coat on the prepared concrete deck, a spray applied or preformed membrane, a protective board, tack coat and then the asphalt applied on top. A schematic of these layers is shown in Figure 7.9.

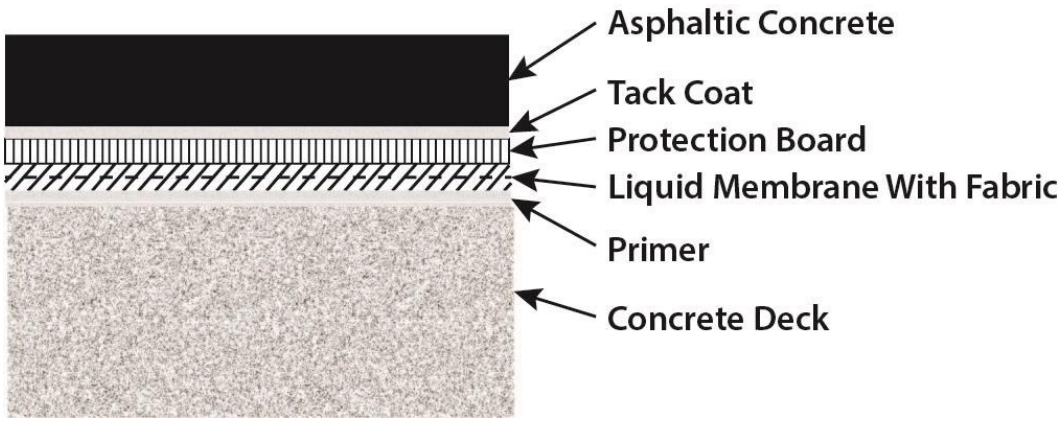

Figure 7.9 Typical Membrane Configuration

7.2.3.2 Concrete Overlays

Rigid concrete overlays are very common on bridge decks and slabs because they have a relatively long life, have quick installation techniques known by most contractors and provide good ride quality when cured. Concrete overlay types include the following concrete variants:

- High performance concrete
- Latex modified concrete
- Silica fume concrete*
- Fly ash modified concrete
- Polymer concretes

*Note that several states have reported bonding issues with silica fume concrete, particularly with patching operations.

Concrete overlays provide a durable wearing surface that stiffens the deck or slab, provides additional cover over reinforcing steel, and reduces permeability of the deck or slab. All of these attributes can slow or eliminate the ingress of water and chlorides that lead to corrosion. Concrete overlays also provide a single step means to patch localized deck spalls and overlay in a monolithic placement of concrete that can lead to improved ride quality.

7.2.3.3 Polymer Overlays

Thin bonded polymer overlays are very common overlays for bridge decks and slabs because they are light weight, result in minimal grade differences, reduce permeability and the bridge can be put back into service quickly. Polymer overlays are typically less than one half inch thick and can be applied in one or more layers. Polymer overlays use resins of epoxy, epoxy urethane, or methyl methacrylate.

A comparison between the overlay options available is presented in Table 7.5:

Table 7.5 Bridge Deck Overlay Summary*

Attribute	Asphalts	Concretes	Polymers
Typical Applications	Thick flexible wearing surface over all common deck materials. With membrane	Durable thicker rigid wearing surface and deck protection	Thinner more flexible deck protection and wearing surface
Typical Thickness Range (inches)	1.5 - 6	1 - 5	0.5 - 1.5
Surface Prep	Clean and milled or roughened	Steel shot, ground or roughened	Steel shot, ground or roughened
Minimum Cure Time for Traffic (hours)	1 - 4	2 - 12	2 - 16
Skid Resistance	Original Surface	Concrete Finish, grind or grooving	Broadcast fine aggregate (sand)
Expected Useful Life (years)	15 - 20	15 - 30	10 - 20

* For more information refer to: International Concrete Repair Institute, Technical Guideline No. 310.2R-2013, *Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair*.

7.2.4 Common Maintenance Activities

7.2.4.1 Bridge Deck Washing

Bridge deck and slab washing is an economical way to remove debris, dirt and chlorides from the roadway surface. Regular flushing can reduce chloride levels in the concrete and improve air circulation and drying. Prior to washing the deck, the local environmental requirements related to the collection of the wash water should be understood. The following procedure can be used for bridge decks and slabs of all materials:

Suggested Procedure

Bridge Deck Washing

1. Sweep the deck to remove accumulated surface debris.
2. Pressure wash the deck at 2700 – 3000 psi (2-3 gallons/minute) starting at the highest point. Water will flow from the high end to the face of lowest rail.
3. Use the pressurized water to flood the deck and push the water, dirt and salt mixture towards the face of the lowest rail (See Figure 7.10).
4. Progress in a back and forth motion, much like mowing a lawn until all the water and debris is along the low rail.
5. Spray clean the rail face or other components that get debris blown onto them.
6. Use water pressure and a vacuum truck to collect all the debris and water from the surface. Local environmental regulations may permit diversion of wash water to the roadway embankment in lieu of collection and disposal.

An example of bridge washing is shown in Figure 7.10.

Figure 7.10 Bridge Washing (Courtesy of DBI Services)

7.2.4.2 Epoxy Crack Injection

Discrete larger deck and slab cracks can be sealed and re-bonded through surface or pressure injection with a material such as epoxy or polyurethane.

Following is a procedure for crack injection:

Suggested Procedure

Epoxy Crack Injection

1. Clean out the cracks to be sealed. Methods to clean cracks include:
   a. Vacuuming
   b. Oil-free compressed air
   c. High pressure water
2. Seal the crack surface. Apply cap sealer epoxy along the dry top surface of the crack and allow it to harden. If the crack has spalled edges or if injection pressures are too high for a surface seal, saw cut the crack first and then, seal the surface by cutting a v-groove and filling it with mortar.
3. Use surface ports or drill injection port holes along the crack.
4. Inject the sealant through the injection ports until the cracks are filled (see Figure 7.11).
5. Allow to cure before returning traffic.

Figure 7.11 Epoxy Injection Repair (Courtesy of Virginia DOT)

For surface sealing with epoxy, the following procedure applies:

Suggested Procedure

Surface Sealing with Epoxy (with Optional Fiber Reinforcement Fabric)

1. Clean out the cracks to be sealed. Methods to clean cracks include:
   a. Vacuuming
   b. Oil-free compressed air
   c. Abrasive blasting (i.e., shot blasting or sandblasting).
2. Apply epoxy onto the clean dry crack on the deck surface.
3. Broom epoxy along the dry top surface of the crack the width of the fiber reinforcement.
4. Roll out the fiber reinforcement fabric onto the wet epoxy surface and press into the epoxy until the fabric is saturated (see Figure 7.12). Overlay as needed for ride quality.
5. Allow to cure before returning traffic.

Examples of surface epoxy sealing with optional fabric are shown in Figure 7.12. Rolling out the fiber reinforcement fabric is shown in Figure 7.12 (left). Detail view of the fabric in-place is shown in Figure 7.12 (right).

Figure 7.12 Surface Epoxy Sealing with Optional Fabric (Courtesy of Virginia DOT)

7.2.4.3 Applying Penetrating Deck Seals

Widespread smaller cracks can be sealed using penetrating sealers to slow the ingress of water and chlorides into the deck or slab.

The following is a procedure for applying penetrating deck sealers:

Suggested Procedure

Application of Penetrating Deck Sealer

1. Repair any spalled or delaminated areas following the procedure provide below. Do not patch surfaces until fully cured.
2. Clean the surface by shot-blasting to remove all contaminants and to open the surface and cracks (see Figure 7.13).
3. Remove dust and debris by blowing off with oil-free compressed air and vacuuming.
4. Protect the expansion joints, and any other areas not to be sealed, from damage during preparation of the surface. The protection must be removed once the epoxy and aggregate have been applied and prior to initial set. Removing the protection must be done soon enough to ensure no harm to the adjacent sealed surface.
5. Mix components per manufacturer recommendations by volume. Mix only the quantity that can be used within its gel time.
6. Apply mixed product in accordance with manufacturer instructions, typically by pouring or spraying on the surface. Distribute material in accordance with manufacturer instructions, which is often evenly with a squeegee, roller or broom, maintaining a liquid head over cracks. Remove all excess material with squeegee or broom. Most products should be dispensed immediately after mixing to ensure adequate crack penetration time (see Figure 7.14).
7. Broadcast fine aggregate (sand). Application of aggregate must be of sufficient quantity to cover the entire surface in excess. No bleed through or wet spots shall be visible. The aggregate should be clean and moisture free. The broadcast of sand will wear off leaving sealed cracks behind (see Figure 7.15).
8. After initial cure, remove excess sand or aggregate by broom or compressed air. Excess sand may reduce surface friction if not removed.
9. When used as primer or pre-treatment prior to overlay consult manufacturer's technical representative for appropriate curing before overlay. Some products are designed to act as a primer for a "wet on wet" application; other products should gel or cure prior to the application of overlays.

Figure 7.13 Shot Abrading Deck Preparation (Courtesy of MDOT)

Figure 7.14 Sealer Application (Courtesy of MDOT)

Figure 7.15 Broadcasting Sand (Aggregate) (Courtesy of MDOT)

7.2.4.4 Full and Partial Depth Concrete Spall Repairs

Deck spalls are a common outcome of cracking, vehicular impact, or reinforcing corrosion in the deck or slab. Concrete patching can be used to repair the spalls. The following procedure will result in a quality patch. Note that if unsound concrete is removed to the extent that the

bottom mat of reinforcement is exposed, stop the patching partial depth procedure and switch to the full depth procedure.

Suggested Procedure

Concrete Spall Repair

1. Identify the deck or slab location to be repaired.
2. Hammer sound or chain the area around the spall to identify and mark adjacent unsound concrete. Mark off square areas outside the limits of the unsound concrete.
3. Combine patches closer than 1 foot into larger patches.
4. Sawcut to a depth of at least 3/4 to 1 inch in a geometric pattern marked in the previous step. Be careful not to cut through reinforcing steel (see Figure 7.16 and Figure 7.17).
5. If the patch is full depth, protect the area under the work area from falling debris. This can be done by placing wooden or metal forms attached to the girders or soffit of the bridge (see Figure 7.18). Forms may be suspended from reinforcing steel by wire ties for areas of less than 3 square feet. In the case of larger openings, forms shall be supported from below by blocking to ensure the form can support the wet concrete weight.
6. Use a lightweight chipping gun (maximum weight of 30 pounds) to chip the concrete out between the edge of the spall and the sawcut lines. Pneumatic hammers shall be worked at an angle of 45 to 60 degrees to the plane of the concrete being removed. Chip the concrete out max aggregate plus 1/4 inch below the top mat of reinforcing steel or any other exposed reinforcing steel layers (see Figure 7.19). All loose concrete within the limits of the patch should be removed. Do not place chipping gun on the reinforcement.
7. Inspect edge of patch to ensure the limits of the delamination have been removed. If a concrete separation is visible (see Figure 7.20), increase the size of the patch.
8. Use sand blasting equipment to remove any rust from exposed reinforcing steel and to clean the exposed concrete surfaces. Touch up epoxy bars.
9. Replace any missing or corroded steel with new bars providing proper overlap lengths. Consult an engineer as needed. Overlap requirements may require a larger area to be chipped out, or drill and epoxy anchor bars, or mechanically splice.
10. Galvanic anodes may be used for additional corrosion protection in the patch region.
11. Check the repair area one more time for any loose concrete or debris inside the spalled area and remove any found. Blow out the repair area with compressed air (see Figure 7.21). Prepared concrete area is shown in Figure 7.22.
12. Apply a bonding agent as specified by the engineer. A bonding agent may not be specified in all cases.
13. Wet the concrete in the patch area but do not allow standing water (Saturated Surface Dry or SSD condition).
14. Mix rapid setting concrete (or conventional concrete) and place concrete in the patch area up to the surface of the surrounding deck (see Figure 7.23). Vibrate the concrete, especially in the corners and around the reinforcing steel (see Figure 7.24).
15. Screed the newly placed concrete to the level of the surrounding deck (see Figure 7.24). Use concrete finishing tools appropriate for the size of the patch.
16. Use a wet broom to provide a roughened surface in a transverse direction to traffic.
17. Cure the concrete per the manufacturer recommendation before restoring service on the bridge (see Figure 7.25). Concrete patches will continue to harden over time but can often support traffic within 2 to 4 hours for rapid setting patch materials.

Figure 7.16 Concrete Repair Photos. Full Depth Repair (left) and Partial Depth Repair (right)

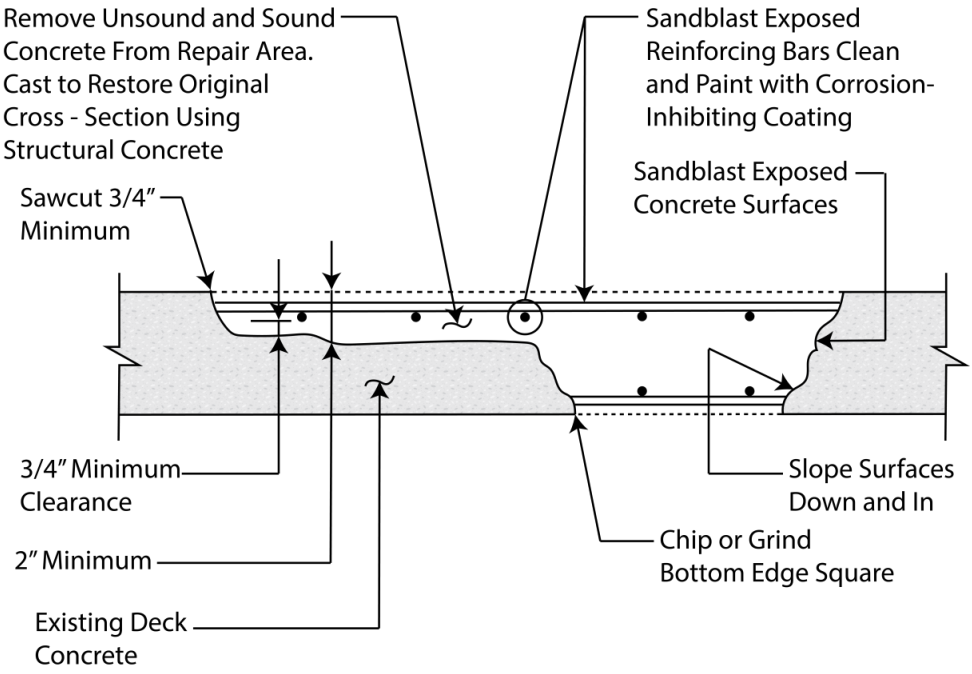
Remove Unsound and Sound Concrete From Repair Area. Cast to Restore Original Cross - Section Using Structural Concrete
Sandblast Exposed Reinforcing Bars Clean and Paint with Corrosion-Inhibiting Coating
Sandblast Exposed Concrete Surfaces
Sawcut 3/4" Minimum
3/4" Minimum Clearance
2" Minimum
Existing Deck Concrete
Slope Surfaces Down and In
Chip or Grind Bottom Edge Square

Figure 7.17 Typical Spall Repair Plan

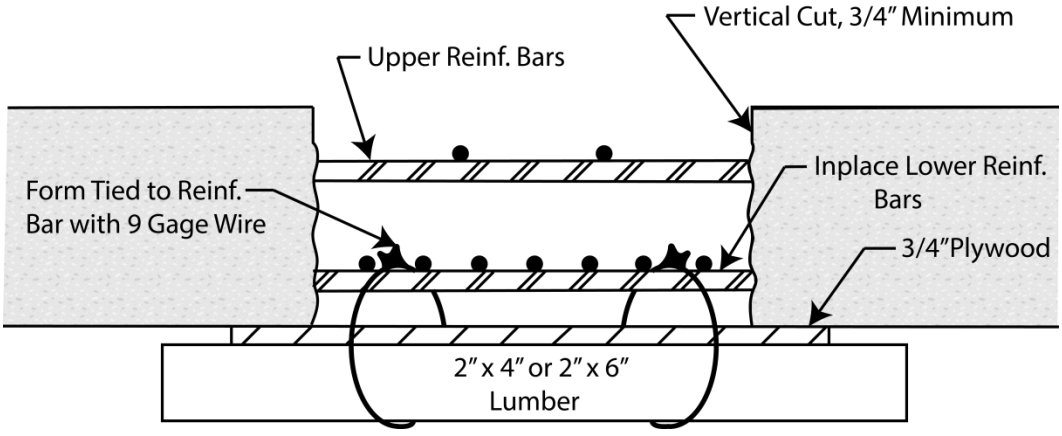
Vertical Cut, 3/4" Minimum
Upper Reinf. Bars
Form Tied to Reinf. Bar with 9 Gage Wire
Inplace Lower Reinf. Bars
3/4"Plywood
2" x 4" or 2" x 6" Lumber

Figure 7.18 Typical Full Depth Repair Forming

Figure 7.19 Concrete Removal (Courtesy of MDOT)

Figure 7.20 Delamination at Edge of Patch (Courtesy of MDOT)

Figure 7.21 Cleaning with Compressed Air (Courtesy of MDOT)

Figure 7.22 Prepared Patch Areas (Courtesy of MDOT)

Figure 7.23 Concrete Placement

Figure 7.24 Vibration and Finishing (Courtesy of MDOT)

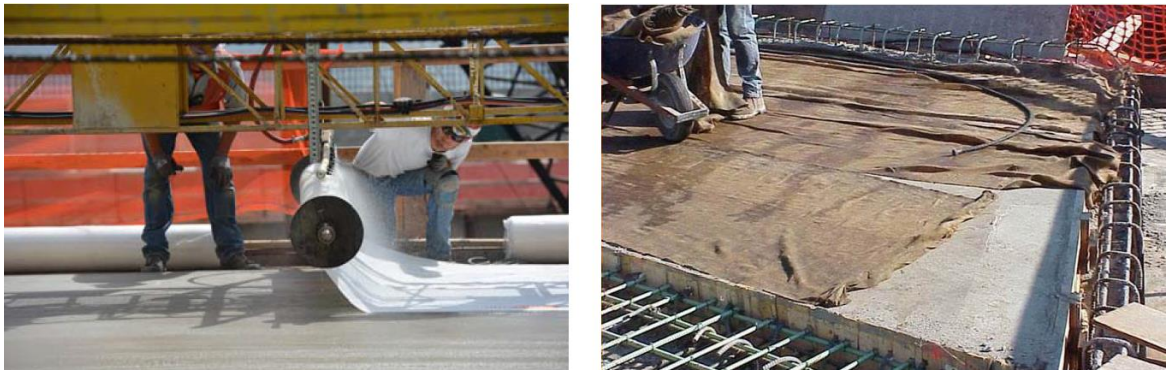

Figure 7.25 Concrete Wet Cure Examples. Wet Cure Blanket (left) and Burlap Sheets (right)

7.2.4.5 Concrete Deck and Slab Delamination Repair

Delamination of a concrete deck or slab is typically caused by corrosion of the reinforcing steel that results in corrosion and expansive carbonation at the rebar level that cracks the concrete along rebar lines parallel to the surface of the deck or slab. Delaminations can be identified through acoustic methods such as chaining or impact echo methods or through density test methods such as ground penetrating radar.

The following procedure can be used to repair deck or slab delaminations:

Suggested Procedure

Deck or Slab Delamination Repair

1. Identify the delaminated deck areas using one of the identified location methods. Deck chaining and hammer sounding are the most common methods (see Figure 7.26).
2. Remove the delaminated concrete areas (see Figure 7.27 and Figure 7.28).
   a. For a small percentage of delaminations, repair can be accomplished following the spall repair procedure.
   b. For larger delamination areas, hydro-demolition equipment should be used to remove the concrete and clean the reinforcing steel of corrosion. Follow steps 3 through 11 for Suggested Procedure: Placement of Concrete Overlay in Section 7.2.4.7. See Figure 7.29 and Figure 7.30.

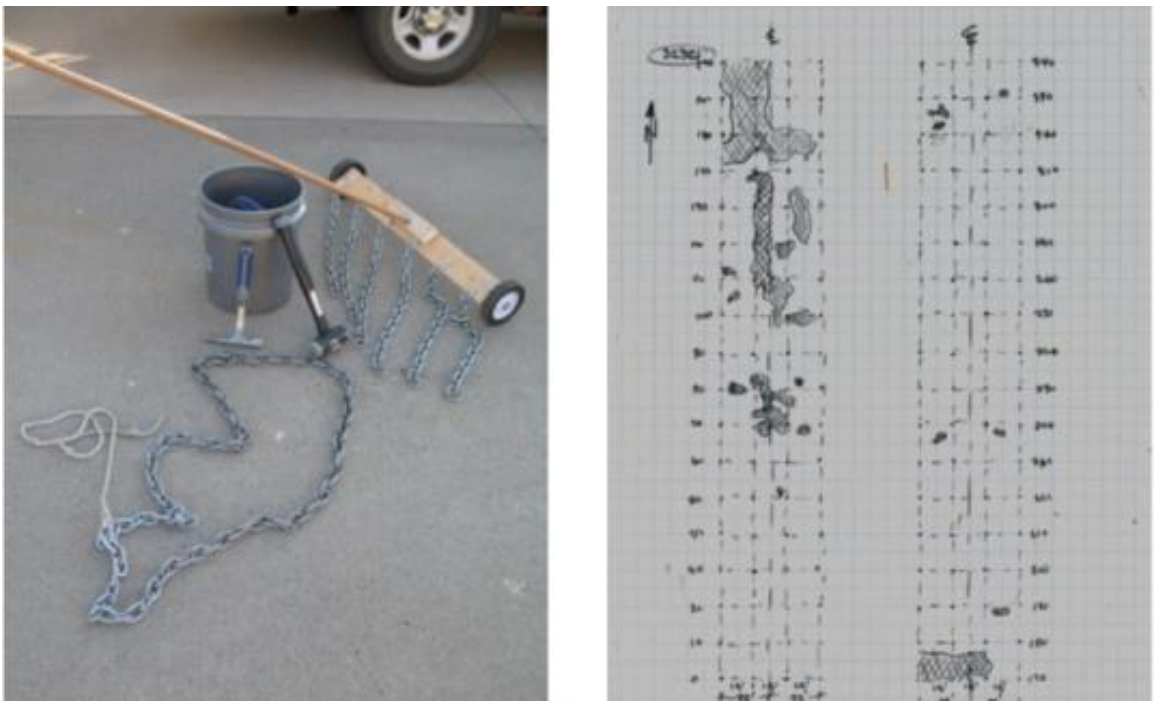

Figure 7.26 Chain Sounding Tools (left) and Delamination Mapping (right)

Figure 7.27 Removing the Delaminated Concrete (Courtesy of Kwikbond Polymers)

Figure 7.28 Sawcut, Chipped Out and Cleaned (Courtesy of Truesdell Construction) Concrete Removal Shown to the Top Mat Only

Figure 7.29 Placing the New Polyester Concrete (Courtesy of Truesdell Construction)

Figure 7.30 Curing the Concrete

7.2.4.6 Placement of Asphalt Overlay with Membrane

Asphalt overlays may be used as an application for waterproofing concrete bridge decks. There are two recommended practices for asphalt overlays on decks.

1. Asphalt overlay with membrane. A membrane is placed, and the asphalt is paved. The membrane is either a pre-formed sheet or a liquid hot-applied rubberized asphalt with polymer resins. See Figure 7.31 for examples of these steps used in placing an asphalt overlay with membrane, where the following are observed: (a) Application of primer to the concrete deck, (b) Laying out the sheet membrane, (c) Heating the sheet membrane with a torch, (d) Sealing the overlap seams with a hand roller, (e) Completed membrane, (f) Compacting the hot mix asphalt.
2. Polymer modified asphalt (PMA). A PMA is paved, and no membrane is used because the PMA has low permeability.

The following suggested procedure applies to asphalt overlay with a membrane.

Suggested Procedure

Asphalt Overlay with Membrane

Before beginning, the deck needs to be swept or air blasted to remove loose material. Any sealers, coatings, or foreign substances that might affect the adhesion or curing of the asphalt or tack coat should be removed.

The steps for applying an asphalt overlay with a pre-formed membrane sheet are:

1. Apply the primer, such as asphalt emulsion.
2. Apply an adhesive, such as oxidized asphalt.
3. Apply an optional ventilating and leveling layer, such as sand asphalt.
4. Place the membrane, lapping the sheets.
5. Place an optional felt protection board.
6. Apply an asphalt tack coat.
7. Apply an asphalt base mixture, which has a life of 30 years.
8. Apply the asphalt surface mixture, which has a life of 10 to 15 years.

Examples of these steps are shown in Figure 7.31.

Polymer Modified Asphalt

The only step with a polymer modified asphalt is to pave the asphalt. Follow the membrane manufacturer recommendations regarding use of a tack coat prior to the placement of a polymer modified asphalt overlay. The most common tack coat used for polymer modified asphalt is a non-diluted asphaltic emulsion.

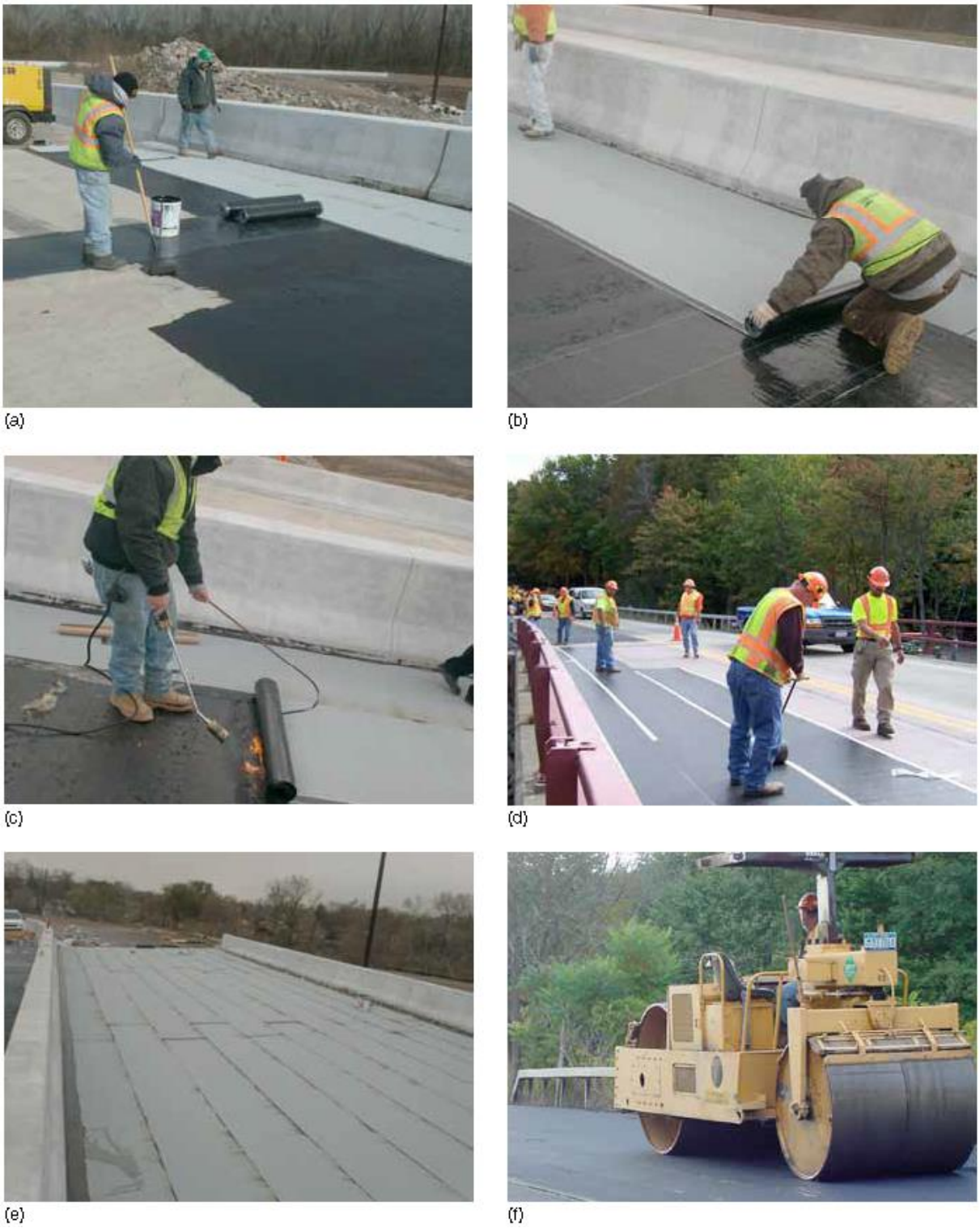

Figure 7.31 Steps in the Installation of a Preformed Sheet Membrane (Courtesy of TRB)

7.2.4.7 Placement of Concrete and Polymer-Modified Concrete Overlay

Overlays can be an effective deck preservation method if the underlying concrete is properly patched and delaminations are addressed prior to the placement of the overlay. Proper hydrodemolition will remove all delaminations. Concrete damaged to or beyond the depth of

the bottom layer of reinforcement requires full depth repairs. Partial and full depth repairs may be poured at the same time as the overlay.

The following procedure applies for all concrete and polymer-modified concrete overlays:

Suggested Procedure

Placement of Concrete or Polymer-Modified Concrete Overlay

1. Remove deteriorated concrete and repair any delaminated or spalled areas. Concrete delaminated or spalled to or beyond the depth of the bottom layer of reinforcement requires full depth repairs. Partial depth delaminations or spalls can be repaired with suitable patch materials and overlaid after appropriate curing. Alternatively, partial depth delaminations or spalls as well as full depth repairs may be poured at the same time as the overlay. See spall and delamination procedure. See Figure 7.27.
2. In preparation for the placement of the overlay, clean the entire surface area of the deck by shot blasting or other approved means (see Figure 7.32) within 24 hours of the overlay placement. Protect metal deck drains and areas of the curb or railing above the proposed surface from the shot blast. The Engineer must approve the use of scarifiers or milling machines in lieu of shot blasting. Hydrodemolition may also be used for to prepare the deck for an overlay.
3. Further clean the deck of any debris by air blasting with dry oil free air or vacuuming. Brooming is not acceptable.
4. Remove all loose concrete, dirt, paint, oil, asphalt, laitance carbonation and curing materials from patches and other foreign material from the surface of the deck.
5. Produce a surface relief equal to the International Concrete Repair Institute (ICRI) Surface Preparation Level 6 to 7 or ASTM E 965 Pavement Macro texture depth of 0.04 to 0.08 inch.
6. Remove any contamination of the prepared deck surface or surface of subsequent courses. Sand blast contaminated areas to produce an acceptable surface for placement of the overlay.
7. Protect deck drains so the overlay material shall not pass into the drains.
8. Rain will not necessarily contaminate the surface. However, care must be taken that no contamination occurs.
9. For conventional concrete overlays, the surface shall be thoroughly water soaked without allowing standing water prior to the placement of the new concrete. For polymer-modified concrete, follow the manufacturer instructions.
10. Place the first course within 24 hours of preparing the deck surface. Deck surfaces exposed for more than 24 hours must be sand blasted and step 3 repeated prior to application of the overlay (see Figure 7.33).
11. Cure overlay as specified.

Figure 7.32 Preparing the Deck with Shot Blasting (Courtesy of Kwikbond Polymers)

Figure 7.33 Placing the Concrete Overlay (Courtesy of Truesdell Construction)

7.2.4.8 Placement of Polymer Overlay

The following procedure applies to epoxy polymer overlays.

Suggested Procedure

Placement of Epoxy Polymer Overlay

1. Remove deteriorated concrete and repair any delaminated or spalled areas. Concrete delaminated or spalled to or beyond the depth of the bottom layer of reinforcement requires full depth repairs. Full or partial depth areas can be repaired with suitable patch materials and overlaid after appropriate curing. See spall and delamination procedure. See Figure 7.27.
2. In preparation for the placement of the overlay, clean the entire surface area of the deck by shot blasting or other approved means (see Figure 7.32 and Figure 7.34a) within 24 hours of the overlay placement, to produce a surface relief consistent with epoxy overlay manufacturer requirements. Protect metal deck drains and areas of the curb or railing above the proposed surface from the shot blast. The Engineer must approve the use of scarifiers or milling machines in lieu of shot blasting. Hydrodemolition may also be used for to prepare the deck for an overlay.
3. Remove any contamination of the prepared deck surface such as traffic striping. Sand blast contaminated areas to produce an acceptable surface for placement of the overlay.
4. Further clean the deck of any debris by air blasting with dry oil free air or vacuuming.
5. Protect deck drains so the overlay material shall not pass into the drains.
6. Deck must typically be dry for a specified period of time in accordance with epoxy manufacturer's recommendations, and must remain dry during the cure period. As such, epoxy overlay should not be placed if rain is forecast before completion of cure, typically 6 to 24 hours depending on manufacturer.
7. Mix epoxy as specified by manufacturer in batches (see Figure 7.34b) that can be mixed and applied while maintaining a "wet edge" for beginning to spread the next batch.
8. Spread the mixed epoxy onto the substrate with a notched squeegee at the recommended rate (see Figure 7.34c and Figure 7.34d). Place the epoxy to permit a continuous operation. Apply the second batch immediately behind the first batch.
9. Broadcast the aggregate immediately (see Figure 7.34e), but stop to maintain a wet edge. Broadcast the aggregate to complete saturation (approximately 1.1 pound per square foot. If wet spots develop, immediately broadcast additional aggregate until a dry surface is re-established. Cure overlay as specified.
10. Once the epoxy has cured, sweep the bridge deck to remove any loose aggregate (see Figure 7.34f).
11. Apply the second coat in the same matter but at a rate that is typically double the rate of the first coat due the roughness of the aggregate. The maximum recoat window is 24 hours.
12. Allow to cure, and sweep any loose aggregate from deck, prior to opening to traffic.

The photo sequence in Figure 7.34 shows several steps in the installation of a polymer overlay (i.e., a two part epoxy resin in this example). The following steps are shown in the figure: (a)

Shotblasting to clean the deck surface, (b) Mixing the two part epoxy, (c) Pouring the epoxy from buckets, (d) Spreading the epoxy with notched squeegee brushes, (e) Broadcasting aggregate onto the epoxy surface, (f) Sweeping up loose aggregate prior to opening the bridge to traffic.

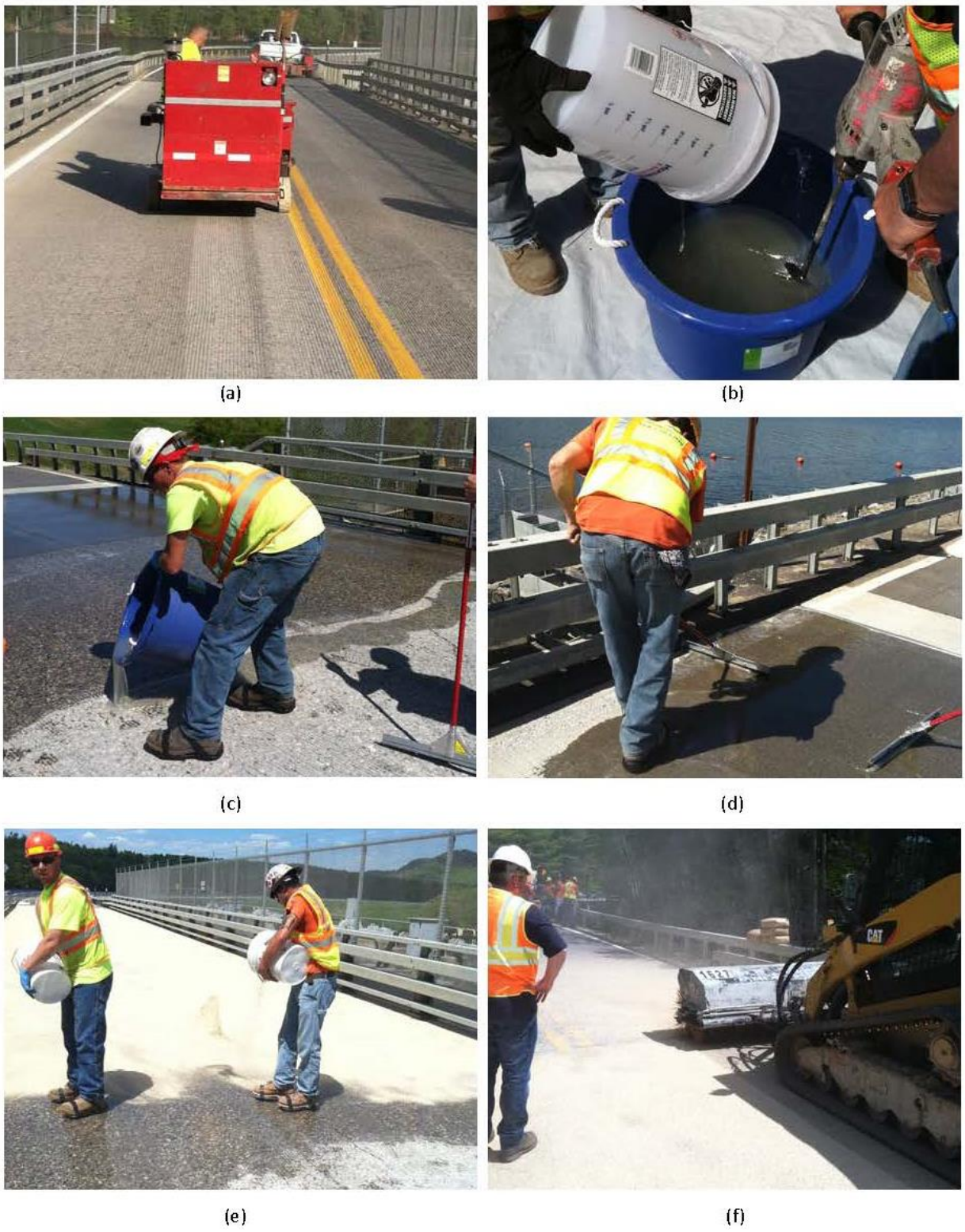

Figure 7.34 Common Steps in Epoxy Polymer Overlay

7.2.4.9 Steel Corrosion Mitigation

Corrosion of reinforced concrete steel is a common problem faced by bridge owners. Strategies to minimize deck steel corrosion have been discussed within this chapter. For decks that already have active corrosion of the reinforcing steel occurring, the following can be used to address the situation:

- Determine if the steel corrosion is in discrete areas or throughout the deck. Delamination surveys can be used to establish the extent of rebar corrosion damage.
- For widespread corrosion damage (greater than 25 percent of the surface area), a partial depth deck replacement should be considered as this repair will remove active corrosion and replace chloride contaminated concrete.
- Hydro-demolition is very effective at removing varying depths of concrete and cleaning the reinforcing steel in preparation for a partial depth deck replacement.

For localized areas of rebar corrosion (see Figure 7.35), the area may be repaired using the procedure presented for spall repair, with the addition of galvanic anodes being attached to the reinforcing in the repair area prior to the placement of the new concrete (see Figure 7.36). The anodes are typically tied on the side of the exposed rebar as close as practical to the surrounding concrete making sure that enough space is left to fully encapsulate the anodes in the repair.

Figure 7.35 Localized Repair Area

Figure 7.36 Placement of Galvanic Anodes (Courtesy of Vector Corrosion Technologies)

For widespread steel corrosion, consideration should be given to adding a cathodic protection system to slow the corrosion of the repaired deck.

7.3 Steel Decks

7.3.1 Introduction

What To Look For

- Cracks
- Broken welds, rivets, bolts
- Failed coatings
- Corrosion
- Section loss
- Loose Elements

Steel bridge decks are common on moveable bridges and other bridge designs that require a lightweight deck. The weight of a steel deck per unit area is much less than that of concrete. This weight reduction of the deck means the machinery that lifts the bridge can be economized. For open grid steel decks, water passing through the deck and onto the superstructure and substructure elements can lead to deterioration. Steel grid decks can be filled or partially filled with concrete to prevent the water from passing through and causing deterioration. Steel bridge decks can be found in design situations where forming concrete is difficult or rapid deck construction is required. In these situations, corrugated, orthotropic or exodermic steel sections may be used by the designer. There are four general types of steel bridge decks commonly encountered:

- Grid decks
- Orthotropic decks

- Corrugated steel decks
- Exodermic Bridge Decks

7.3.1.1 Grid Decks

Welded grid decks have their components welded together in various patterns (see Figure 7.37). These components consist of bearing bars, cross bars, and supplementary bars. The bearing bars support the grating. Bearing bars are laid on top of the beams or stringers perpendicularly and are then field-welded or bolted to the superstructure.

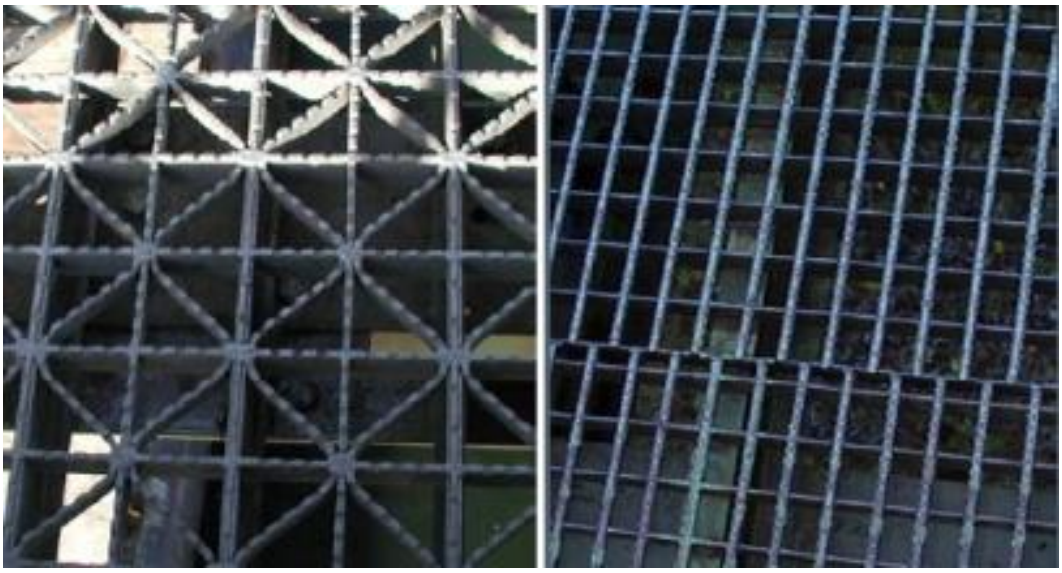

Figure 7.37 Open Grid Decks. Diamond (left) and Rectangular (right)

A riveted grid deck consists of bearing bars, crimp bars, and intermediate bars (see Figure 7.38). Bearing bars run perpendicular to the superstructure and are attached to the beams or stringers by either welds or bolts. Crimp bars are riveted to the bearing bars to form the grating. Intermediate bars are parallel to the bearing bars but are not as long. The crimp bars are riveted to intermediate bars.

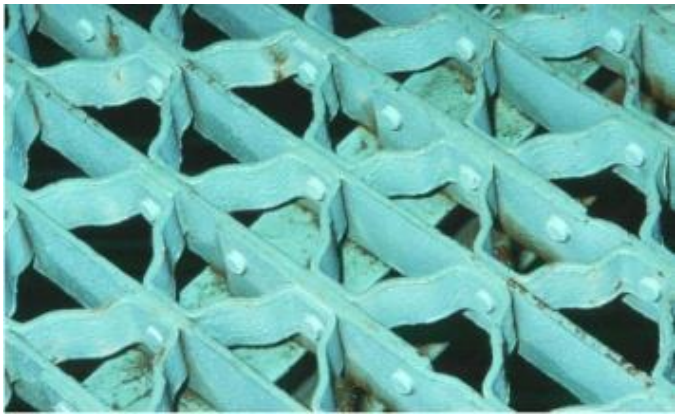

Figure 7.38 Riveted Steel Grid Deck

Welds and rivets used to construct steel grid decks have a long history of cracking from fatigue or deterioration caused by vehicle tires. To minimize cracking, steel grid decks have been

fabricated with slotted holes to eliminate the use of welds or rivets (see Figure 7.39). Transverse distribution bars are inserted into the slots rotated into position and locked into place without the use of any welds or rivets.

Figure 7.39 Slotted Grid Deck

Concrete filled grid decks are commonly used to provide added protection from grid damage and to protect the superstructure and substructure from damage caused by surface water and chlorides. An example of a concrete filled grid deck is shown in Figure 7.40. Filling the grids with concrete can add considerable weight to the deck. The weight of filling the grids can be limited by filling portions of the grids that are in the wheel lines or near bearings or other structural components that need added protection from the elements.

Figure 7.40 Filled Grid Deck (Courtesy of Florida DOT)

7.3.1.2 Orthotropic Decks

An orthotropic deck consists of thin steel plate stiffened by a series of closely spaced ribs placed perpendicular to their supporting members (see Figure 7.41). The deck acts integrally with the steel superstructure. An orthotropic deck becomes the top flange of the entire floor system.

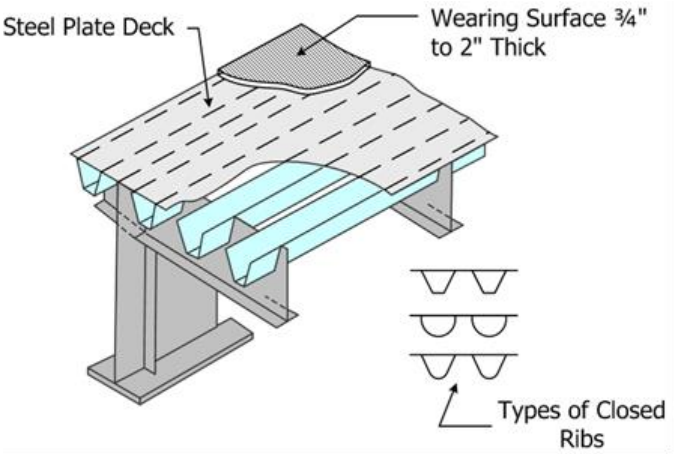

Figure 7.41 Orthotropic Bridge Deck

7.3.1.3 Corrugated Decks

Corrugated steel flooring is popular because of its light weight and high strength. Corrugated decks consist of thin steel plate material bent into ribs and covered by a layer of bituminous wearing surface or polymer concrete. A schematic of the corrugated deck surface is shown in Figure 7.42. The corrugated flooring spans transversely between the supporting superstructure members and it typically welded to steel members or nailed or bolted to timber and concrete superstructures. The wearing surface thickness varies from the centerline of the deck to the edge of the roadway to achieve proper cross slope. There are no reinforcement bars utilized in this deck type.

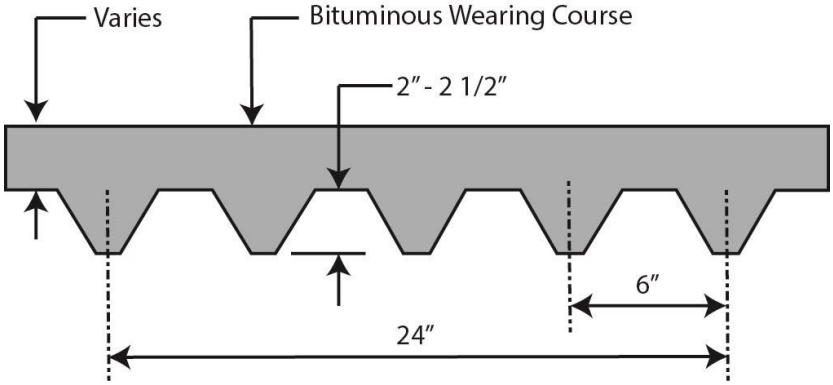

Figure 7.42 Corrugated Bridge Deck

7.3.1.4 Exodermic Bridge Decks

Exodermic decks are a variant of the tradition grid deck that use steel grids with concrete attached to the superstructure. Composite action is achieved by studs that extend into the reinforced concrete deck and are welded to the grid deck below (see Figure 7.43). Galvanized sheeting is used as a bottom form to keep the concrete from falling through the grid holes. Exodermic decks generally weigh 50 to 65 percent less than precast reinforced concrete decks.

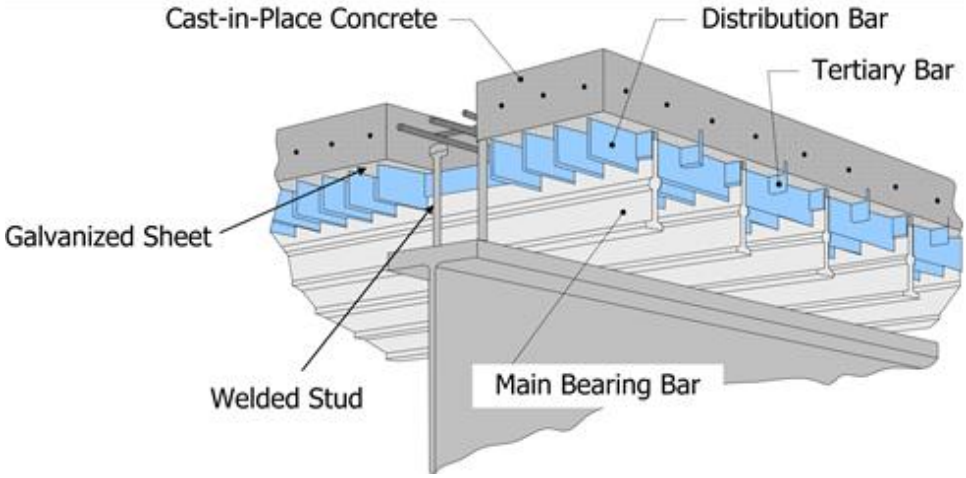

Figure 7.43 Exodermic Bridge Deck

7.3.2 Deck Protection Methods

Steel decks are subject to corrosion that can be minimized using many of the same techniques used to control reinforced concrete rebar corrosion. These techniques include removing debris and chlorides through regular flushing, applying protective coatings, and utilizing cathodic protection systems.

7.3.2.1 Washing and Cleaning

Bridge deck and slab washing is and economical way to remove debris, dirt and chlorides from the roadway surface. Regular flushing can reduce chloride levels and improve air circulation and drying.

7.3.2.2 Coating Systems – Paint and Galvanization

Paints provide protection from moisture, oxygen, and chlorides. Usually three coats of paint are applied. The first coat is the primer, the next is the intermediate coat, and the final coat is the topcoat. Various types of paint are used, such as oil/alkyd, vinyl, epoxy, urethane, zinc-rich primer, and latex paints.

Galvanizing can be used to protect steel decks. The galvanized coating retards the corrosion process and lengthens the life of the steel deck. The two dissimilar metals form an electrical current which reduces the corrosion in the steel. There are two methods of galvanizing steel decks (shop applied and field applied). Hot-dipping the steel deck member usually takes place at a fabrication shop prior to the initial placement of the steel deck. Steel sections are placed into a molten bath of zinc fusing the zinc to the surface of the steel.

When sections of the deck are too large or when field coating is required, the metallizing process is used. Metallizing is a thermal spray of zinc protective coating for steel. Zinc or aluminum wire is continuously melted in an electric arc spray or gas flame spray gun. Droplets of molten metal from the wire are deposited onto the steel forming the protective coating. This sprayed metal coating is both a barrier coating and a galvanic coating in one. The coating provides protection to the steel similar to galvanizing. Metallized coatings often have a top coat (sealer) to extend service.

7.3.2.3 Cathodic Protection

Steel bridge decks are subject to corrosion that can be minimized through the use of cathodic protection systems. Both sacrificial anodes and active current systems can be designed to control the rate of corrosion. These systems should be designed by a corrosion professional.

7.3.3 Common Maintenance Activities

7.3.3.1 Pressure Washing

Pressure washing will remove debris and chlorides from the steel and promote air circulation and drying. The following procedure can be used for bridge decks and slab of all materials:

Suggested Procedure

Pressure Washing Decks and Slabs

1. Sweep the deck to remove accumulated debris.
2. Pressure wash the deck starting at the highest point. Follow concrete deck washing procedures.
3. Use the pressurized water to flood the deck and push the water, dirt and salt mixture towards the face of the lowest rail.
4. Progress in a back and forth motion, much like mowing a lawn until all the water and debris is along the low rail.
5. Spray clean the rail face or other components that get debris blown onto to them.
6. Use water pressure and a vacuum truck to collect all the debris and water from the surface.

7.3.3.2 Repairing Broken Connections

The welds or rivets that join the deck grids and hold them down may break over time (see Figure 7.44). It may be necessary to reinforce or replace damaged grid plates. When several plates have been damaged or dislodged it is best to cut out and replace that portion of the deck.

When to Call the Engineer

Call the Engineer before cutting out parts of deck.

Figure 7.44 Typical Broken Grid Section

The following procedure can be used to repair a broken grid section:

Suggested Procedure

Broken Grid Section Repair

1. Closely inspect the damaged area around the broken members for additional cracking or bent grid sections.
2. Determine a rectangular section that will address all localized damage.
3. Cut the grid section out within the identified repair area.
4. Cut a replacement grid section to fit within the newly created repair area.
5. Weld the repair section into the prepared hole in the grid deck. Welding should be done on both sides of every grid member.
6. Protect the newly welded connections with zinc rich primer or by metalizing.

7.3.3.3 Filling the Deck with Concrete

Another method of repair commonly used is to fill or partially fill the steel grid with concrete or lightweight concrete. The engineer should evaluate the effects of the additional dead load from the concrete prior to placing. The additional dead load may require rebalancing of moveable bridges. The concrete filled grid acts as a reinforced concrete deck. The steel grid provides the steel reinforcement and the concrete fill provides stiffness to help carry the load. This repair technique has several advantages including: increased deck strength, reduction in the effect of weathering by limiting the water penetration, support of the welded grid joints, and increased skid resistance.

When to Call the Engineer

Call the Engineer before adding dead load to the bridge.

7.3.3.4 Mitigating Steel Corrosion

Steel deck corrosion is primarily mitigated through the use of surface protection system such as paint or galvanization. Paints can be applied to deck sections that are properly prepared using the following steps:

Suggested Procedure

Paint Application

1. Some states recommend pressure washing for chloride removal prior to proceeding with SSPC surface preparation guidelines.
2. Allow the steel to fully dry.
3. Apply zinc rich primer, followed by base and top coat applications of paint.
4. Grid decks can be difficult to paint due to the number of edges and tight angles. Special care should be taken to adequately cover all areas.

For spray applied zinc (metallizing), consult a professional. Light sand blasting may also be in order if there is corrosion present.

7.3.3.5 Restoring Surface Friction

Both open grids and concrete-filled grids without a wearing surface are subject to decreased skid resistance over time.

Open Grid Decks - Some agencies apply studs to the deck surface to improve traction. The studs are typically 5/16 inches in diameter and 3/8 inches high and welded on intersections of the cross members to provide surface friction.

Filled Grid Decks - Filled grids can develop cupping or wear of the concrete between the grid bars, which exposes the grid to direct wheel loads. The surface of the filled grid then becomes similar to that of an open grid, and skid resistance quality declines. In wet weather, this is dangerous as water is held in the cups. In freezing weather the hazard increases due to ice formation. This condition is best addressed through the application of a thin bonded polymer overlay. See Section 7.2 of this chapter.

7.4 Timber Decks

7.4.1 Introduction and Deck Types

What To Look For

- Decay
- Crushing of wood
- Loose connections
- Loose wood members
- Cracks in wood
- Cupping, curling of planks

Timber can be desirable for use as a bridge decking material because it is resistant to deicing agents that can deteriorate concrete and steel decks. Timber is abundant in parts of the country, is easy to fabricate in any weather condition, it is light weight, and it is a renewable material source.

Timber deck performance is directly related to the ability to control moisture and pest infestation. Moisture fluctuations in timber members can lead to decay or crushing of the member. Controlling the moisture is the simplest, most economical method of slowing decay in timber bridge decks. When exposure to wetting is reduced, members can dry to moisture contents below that which will support most fungal and insect growth. Moisture control was the only method used for protecting many covered bridges constructed of untreated timber, some of which have provided service lives of 100 years or more. Moisture control involves a common sense approach of identifying areas with visibly high moisture, locating the source of water, and taking corrective action to eliminate the source. For example, drainage patterns on approach roadways can be rerouted to channel water away from the bridge rather than onto the deck. Cleaning dirt and debris from the deck surface and drains also reduces moisture and improves air circulation limiting decay.

Timber decks are classified into four major types (plank systems, nail laminated, glue laminated, and stress laminated) as described in the following sections.

7.4.1.1 Plank Systems

Plank decks consist of timber planks laid transversely across the bridge on their side (see Figure 7.45). The planks are individually attached to the bridge beams using spikes or bolt clamps.

Figure 7.45 Plank Decking

7.4.1.2 Nail Laminated

Nailed laminated decks consist of timber planks with the wide dimensions of the planks in the vertical position and laminated by nailing to the adjacent planks (see Figure 7.46). On timber beams, each lamination is toenailed to the beam. On steel beams, clamp bolts are used as required. In either case, laminates are generally perpendicular to the roadway centerline.

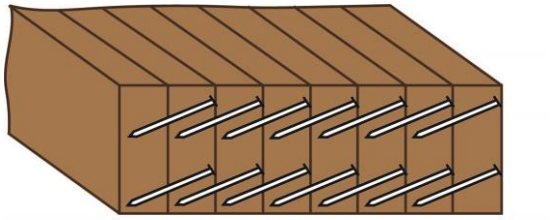

Figure 7.46 Nail Laminated Deck Cross Section

7.4.1.3 Glue-Laminated (Glulam)

Glulam is an engineered wood product in which pieces of sawn lumber are glued together with waterproof adhesives. Glue-laminated deck panels come in sizes usually 4 feet wide. The panels are laid transverse to the traffic and are attached to the superstructure. In some applications, the panels are interconnected with dowels. There are several techniques used to attach glue-laminated decks to the superstructure or a floor system, including nailing, bolting, reverse bolting, clip angles and bolts, and nailer planks (see Figure 7.47).

Figure 7.47 Glulam Members Attached with Metal Clips

7.4.1.4 Stress-Laminated

Stress-laminated decks are constructed of sawn lumber glulam wood post tensioned transversely utilizing high strength steel bars. Stressed timber decks consist of thick, laminated timber planks which usually run longitudinally in the direction of the bridge span. The timber planks vary in length and size. The laminations are squeezed together by high strength steel bars that are passed through predrilled holes in the laminations (see Figure 7.48). Steel channel bulkheads or anchorage plates are then used to anchor the bars on the sides of the bridge. This stressing operation creates a friction connection between the laminations, thereby enabling the laminated planks to span longer distances. Stress-laminated decks can be used as the superstructure for shorter span bridges.

Figure 7.48 Stress-Laminated Deck with Stressing Bolts Shown

Regardless of the design type, timber decks and slabs are subject to decay and infestation if not properly maintained. Nailed and laminated decks are also subject to loosening of the laminations over time.

7.4.2 Protection Methods - Timber Treatments and Preservatives

Protective systems are necessary to resist decay and extend the useful life of timber bridge decks. Common timber deck protection methods include surface applied and impregnated chemicals and sealants. The most common timber protection treatments are:

- Water repellents

- Timber Preservatives
- Fumigants
- Fire retardants
- Paints

7.4.2.1 Water Repellents

Water repellents help to prevent water absorption in timber decks which will lead to slow decay from molds and weathering. The amount of water in wood directly affects the amount of expansion and contraction of the member. Water repellents are used to lower the absorbed water content of timber deck members and will need to be reapplied periodically. Because it needs to be applied rather frequently, it is not the best means of protecting timber structures. Care should be taken when using water repellents on bridge decks to avoid reducing the surface friction of the deck.

7.4.2.2 Preservatives

Timber preservatives are usually applied by pressure, which forces the preservative into the timber deck member (see Figure 7.49). Surface applied preservatives are also available. Preservatives are the best way to protect against decay.

Figure 7.49 Typical Pressure Treated Markings (Courtesy of Michael B. Johnson)

Preservatives protect the timber from moisture penetration, fungus development and pest infestation. Preservatives are either oil-based or water-based. Some common oil-based preservatives are:

- Coal-tar creosote - Coal-tar creosote poses potential health concerns and is no longer used in new construction, but can be readily found in existing bridge decks. Care should be taken when burning or cutting creosote to not breathe in the smoke or dust.
- Pentachlorophenol - Pentachlorophenol is used as an above-ground decay inhibitor.
- Copper naphthenate - Copper naphthenate can be used for above-ground, ground contact and only freshwater applications
- Oxine copper - Gloves and dust masks should be used when working with wood treated with oxine copper.

Common water based preservatives include:

- Chromated copper arsenate (CCA) - CCA was the most popular preservative used from the late 1970's until 2004
- Ammonical copper zinc arsenate (ACZA) – No longer available in the United States
- Alkalkine copper quaternary (ACQ) – Newer preservative to replace CCA.
- Copper azole - Newer preservative to replace CCA.

Surface treatments are applied to existing bridge members to protect newly exposed, untreated wood from decay or to supplement the initial treatment some years after installation. This type of treatment is most effective when applied before decay begins and is commonly used for treating checks, splits, delaminations, mechanical damage, or areas that were exposed during construction. The ease of application and effectiveness of surface treatments make them useful in preventive maintenance, however the shallow penetration limits their effectiveness against established internal decay.

Conventional liquid wood preservatives are applied by brushing, squirting, or spray-flooding the wood surface. Creosote used to be the most commonly used preservative, but penta and copper naphthenate are used more commonly today. The wood surface should be thoroughly saturated with preservative so that all crevices are treated.

The effectiveness of surface treatments depends on the thoroughness of application, wood species, size, and moisture content at the time of treatment. Wet wood absorbs less preservative than does dry wood. This factor is significant in timber bridges because many areas requiring treatment are subject to wetting. Tests indicate that improved treatment of wet wood was obtained by using preservatives at double the normal 3- to 5-percent concentration. Although field tests show that surface treatments in above ground locations can prevent decay infections for up to 20 years or more, it is recommended that treatments used for bridge applications be systematically reapplied at intervals of 3 to 5 years to ensure adequate protection from decay.

7.4.2.3 Fumigants

Fumigants are specialized preservative chemicals that are placed in pre-bored holes to control internal decay. Over a period of time, the fumigants volatilize into toxic gases that move through the wood, eliminating decay fungi and insects. Fumigants can diffuse in the direction of the wood grain for 8 feet or more from the point of application in vertical members, such as posts. In deck and slab members, the distance of movement is approximately 2 to 4 feet from the point of application.

The three chemicals most commonly used as liquid fumigants are Vapam, Vorlex and chloropicrin. Solid fumigants are available in capsules. These fumigants are very hazardous and should be handled with extreme care utilizing full personal protective equipment. Solid fumigants provide increased safety, reduce the risk of environmental contamination, and permit fumigant use in previously restricted applications.

To be most effective, fumigants must be applied to sound wood (free of decay or infestation). When applied in very porous wood or close to surfaces, some of the fumigant is lost by diffusion to the atmosphere. Before applying fumigants, the condition of the member should be

carefully assessed to identify the optimal boring pattern that avoids fasteners, seasoning checks and decayed structure.

7.4.2.4 Fire Retardants

Fire retardants slow the spread of fire and prolong the time required to ignite the wood. The fire retardant can be applied under pressure at a wood treating plant or can as a surface coating. In both cases, the treatment provides a physical barrier slowing the spread of flames. The treated wood chars but does not oxidize slowing the progress of fire to the material. There are several commercially available wood-based construction materials using pressure-treatment sold under trade names of “Dricon”, “D-Blaze”, and “Pyro-Guard”, as well as factory-applied coatings under the trade names of “PinkWood” and “BluWood”.

7.4.2.5 Paints

Paints applied to timber surface can help to keep some moisture out of timber members if all areas of the members can be completely coated. As the timber members swell and shrink with seasonal changes, the paint systems will generally begin to crack and allow additional moisture into the wood that will accelerate the breakdown of the painted coating and lead to decay in the timber member. For paints to be effective, they must completely cover the surface area of the timber and must be reapplied regularly. Tire wear on bridge decks and slabs will quickly wear away and applied paints in the wheel lines limited the usefulness of paints on the surface.

7.4.3 Timber Deck Overlays

Timber decks are often overlayed with a wearing surface to protect the wood from abrasion and to better distribute the load imparted by vehicle tires. Timber decks have similar overlay options to other materials with a few exceptions. Timber decks often have timber wearing surfaces or “runners” applied to protect the underlying deck members (see Figure 7.50). Timber runners are typically placed perpendicular to the deck timbers along the length of the bridge in the normal traffic wheel lines. Timber runners offer the benefit of better distributing the vehicle load and protecting the underlying deck from abrasion from vehicle tires. Timber runners are easily removed and replaced when they wear out.

Figure 7.50 Typical Timber Deck Runners

Other common overlays for timber decks include asphalt, concrete and polymer overlays. See Section 7.2 for more information of these overlays.

Timber members can deteriorate more rapidly if the wood remains wet for long periods of time. To minimize trapped moisture on timber decks, most overlays utilize a waterproofing membrane under the wearing surface material. Section 7.2 covers typical membrane systems. The application of the waterproofing membrane to a timber deck can be accomplished using the procedure presented in the common maintenance activities portion of this section.

7.4.4 Common Maintenance Activities

7.4.4.1 Washing and Cleaning

Bridge deck and slab sweeping and washing is and economical way to remove debris and dirt from the timber members and joints between members. Regular washing can improve the air circulation and drying around the member reducing the moisture content and rate of decay. The following procedure can be used for timber bridge decks and slabs:

Suggested Procedure

Washing Timber Bridge Decks and Slabs

1. Sweep the deck to remove accumulated debris.
2. Pressure wash the deck starting at the highest point. Use the water pressure to blast out the crevices and butt edges around the timber members.
3. Use the pressurized water to flood the deck and push the water and dirt towards the face of the lowest rail or off the bridge if permitted.
4. Progress in a back and forth motion, much like mowing a lawn until all the water and debris is along the low rail or off the bridge.
5. Spray clean the rail face or other components that get debris blown onto to them during the operation.
6. Use water pressure and a vacuum truck to collect all the debris and water from the surface or blow it off the bridge if allowed by local environmental regulations.

Note that pressure washing timber may not be appropriate, as high pressure can abrade or roughen the surface, as well as force water into the cracks and checks.

7.4.4.2 Treating Timber Deck and Slab members with Fumigants and Preservatives

Surface treatments are applied to existing bridge members to protect newly exposed, untreated wood from decay or to supplement the initial treatment some years after installation. This type of treatment is most effective when applied before decay begins and is commonly used for treating checks, splits, delaminations, mechanical damage, or areas that were exposed during construction.

7.4.4.3 Applying Water Proofing and Surface Preservatives

Suggested Procedure

Applying Waterproofing and Surface Preservatives

1. Clean and dry the timber area to be treated. Special attention should be placed on sawn ends, splits, checks and timber sections that touch the soil.
2. Apply the desired sealer or preservative to the surface of the wood by brush or sprayer. The wood surface should be thoroughly saturated with preservative so that all crevices are treated.
3. Allow the sealer or preservative to penetrate the surface of the wood.
4. Reapply at intervals of 3 to 5 years to ensure adequate protection from decay.

7.4.4.4 Using Fumigants

Fumigants are specialized preservative chemicals that are placed in pre-bored holes to prevent or arrest internal decay. Over a period of time, the fumigants volatilize into toxic gases that move through the wood, eliminating decay fungi and insects.

Suggested Procedure

Fumigant Application

1. Identify the timber members that require fumigant treatment
2. Bore holes in the timber as specified by the fumigant manufacture, typically 2 to 4 feet apart for bridge deck and slab members. Avoid splits, knots and fasteners.
3. Inject liquid fumigant (Vapam, Vorlex or chloropicrin) or solid fumigant capsules into the bored holes. Solid fumigants provide increased safety, reduce the risk of environmental contamination, and permit fumigant use in previously restricted applications.
4. Seal the bored holes with treated wood plugs.

7.4.4.5 Replacing Deck Planks

Maintenance crews can easily replace a timber plank deck and this may be the most cost effective alternative when the deck has deteriorated to the stage that repeated plank replacements would be necessary. Following is a procedure for replacing plank decks:

Suggested Procedure

Replacing Deck Planks

1. Remove and store deck-mounted curbs, parapets, wheel guards and railings.
2. Remove existing deteriorated planks that can be repaired or replaced in the same day.
3. Clean and apply preservative to the top flanges of stringers as described above.
4. Place new planks. Planks should be laid with the heart side down because it is more resistant to decay. ¼-inch spacing should be provided between planks for drainage, expansion, and air circulation. Structural grade hardwood planks, 6 to 10 inches wide, are preferred as decking material because wider planks have a tendency to curl. Planks of the same thickness should be placed adjacent to each other with a full, even bearing on the stringers. Wedges should not be used to level flooring be-cause they are easily dislodged and leave the flooring in a loose and uneven condition.
5. Planks are attached to supporting beams with 1/4 to 1/2 inch diameter galvanized steel spikes or stainless steel screws (recommended), approximately twice as long as the deck planks are thick. If spikes are used, resistance to withdrawal is improved if spikes are driven at a slight angle rather than perpendicular to the beam. Where space allows, two screws/spikes are typically placed in each plank at each beam.
6. Apply wood preservative to the ends of any plank that is field cut.
7. Apply a membrane and wearing surface. (Optional)
8. Replace deck-mounted appurtenances.
9. If this requires drilling into the planks, then treat the holes with fumigants or preservatives before installing the hardware.

7.4.4.6 Installation of a Waterproofing Membrane and Asphalt Overlay

The following procedure can be used to effectively apply a water proofing membrane to a timber deck for an asphalt overlay. Overlays of concrete or polymers will follow a similar step with the exception of the tack coat material.

Suggested Procedure

Applying a Waterproofing Membrane to a Timber Deck

1. Repair any split, broken or decayed timber deck members.
2. Clean the deck surface completely with compressed air and vacuums.
3. Apply a layer of hot mix asphalt concrete over the deck 1 ½ to 2 inches thick (treated wood) or liquid asphalt sprayed directly onto a bridge deck (untreated wood). A timber bridge deck treated with creosote, penta, or copper naphthenate carried in a heavy oil solvent already has an oily surface. The asphalt emulsion will not penetrate the timbers well. A diluted application rate of 0.05 to 0.10 gallons of asphalt per square yard is more appropriate for a treated timber bridge deck.
4. Roll membrane starting at the lowest points and working toward the highest points so that water would drop off the higher layer without getting under the membrane like the shingles on a roof. A paving membrane is a fabric with polymer-modified (or rubberized) asphalt on one or both sides. This asphalt melts when hot asphalt is applied over it. The melted paving membrane provides a waterproof layer between the pavement overlay and the underlying structure.
5. Roll the deck with pneumatic roller to insure good contact and adhesion to the pavement surface. Bubbles and wrinkles should be slit and smoothed.
6. Trowel a bead of rubberized mastic or other suitable sealing compound along the edge of the membrane at the curb.
7. Apply a standard pre-paving tack coat usually asphalt emulsion diluted one to one with water, applied to ensure a bond between the surface being paved and the overlying asphalt pavement course.
8. Place the hot asphalt concrete wearing surface according to the project specifications. When the hot asphalt is placed on an asphalt-impregnated paving membrane the membrane immediately melts, forming a pool of liquid asphalt between the deck and the asphalt pavement.

7.5 Fiber Composite Decks

7.5.1 Introduction

Fiber Reinforced Polymer (FRP) is a newer bridge material that is becoming more popular for use in bridge decks and slabs. FRP is growing in use because this material offers several advantages over more traditional bridge deck materials:

- FRP is very light
- FRP is very strong
- FRP will not corrode

FRP deck sections are particularly well suited for moveable bridges and for decks and slabs subject to highly corrosive environments.

Modern FRP composite decks are typically made of pultruded sections (e.g., honeycomb shaped, trapezoidal, or double-web I-beams). Examples of open honeycomb FRP composite decks are shown in Figure 7.51. Slabs are often made using a vacuum assisted process.

Figure 7.51 Open Honeycomb (Left and Right)

Solid core sandwich decks contain foam or other fillers at the core. This type of deck is manufactured by using a process called Vacuum-Assisted Resin-Transfer Molding (VARTM); Examples are shown in Figure 7.52.

Figure 7.52 Solid Core Sandwich (Left - Single Grid, Right - Double Grid)

Hollow core sandwich decks consist of deck sections that contain pultruded shapes that are fabricated together (see Figure 7.53).

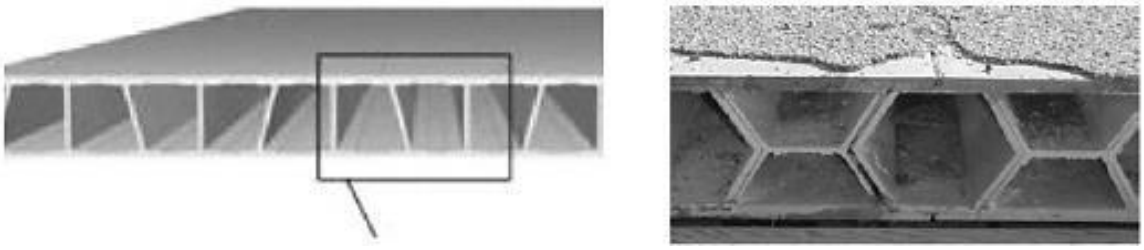

Figure 7.53 Hollow Core (Left - Rectangular Grid, Right - Hexagonal Section)

7.5.2 Deck Protection

FRP deck and slab sections are almost always covered with a wearing surface material to provide enhanced friction and fire protection of the FRP. FRP can lose strength when exposed to high heat that could result from vehicle accident and resulting fire. Wearing surfaces can be asphalt, concrete based products or polymer overlays. Concrete and polymer wearing surfaces also provide protection from chemical attack of the FRP in deicing environments. Wearing surfaces on FRP decks can be repaired using the common methods used with other deck and

slab materials. Care should be taken during the wearing surface repair to avoid saw cutting or chipping into the FRP deck surface.

Call the engineer if any of the following situations are observed in the field:

- Delaminations between fiber layers of the section
- Blistering on the surface or debonding of the material
- Cracking or fiber exposure
- Voids, wrinkling or discoloration

7.5.3 Repairs

FRP deck and slab sections can be constructed of a variety of fiber material and resins. The most common systems are Carbon, Glass, and Aramid FRP. Common resins used in FRP decks and slabs include epoxies, polyesters and vinyl esters. Because of the variety of resins and fibers used in the fabrication of FRP sections, repair of FRP sections is not advised for bridge maintenance crews unless they have had special training in the repair of FRP and are capable of wet lay-up repairs. Wet lay-up repairs for decks and slabs have been shown to restore load capacity to the section; however the durability of wet lay-up repairs is not well documented.

Emergency repairs of localized deterioration of an FRP deck or slab sections can be accomplished with the use of steel plating to cover a small deck delamination or similar deterioration until the engineer can assess the situation.

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Chapter 8 - Deck Expansion Joints

8.1 Purpose and Significance of the Expansion Joint

The purpose of an expansion joint on a bridge is to provide a gap for bridge movement with a smooth riding surface for motorists. The primary movement of the bridge is related to temperature changes; bridges will expand in warm weather and contract in cool weather. There are secondary movements associated with the material properties of the bridge components, such as creep and shrinkage. A bridge also moves under live load.

Bridge deck joints often refer to the system that spans the opening to provide a smooth riding surface over the gap, and protects the portion of the bridge directly below the opening; these types of joint systems are called closed joints. Open joints were often used in older bridges and they frequently allowed drainage to flow through the joint and down onto bridge components below. However, sometimes open joints relied upon drainage troughs below the joints to protect the bridge beneath from debris and water. Newer flexible materials used in the last 50 years have made open joints rare, and many have been retrofitted to include materials which allow for movement but prevent water and debris from flowing freely through the opening. Fixed joints which do not provide for movement should also provide a smooth riding surface and protect the portion of the bridge directly beneath it.

Faulty bridge deck joints contribute to the deterioration of every major bridge component (deck, superstructure, and substructure) by allowing corrosive deicing salts, water and debris through the gap. Deck ends, beam ends, abutment seats, pier caps and bearings can be at risk for serious deterioration through this exposure. The significance of keeping bridge joints in a good state of repair cannot be overemphasized.

The purpose and significance of expansion joints should be at the forefront of the maintenance of these bridge elements. Generally, none will be completely watertight, so vigilance in assessing, maintaining, and replacing deck joints is key to keeping bridges in a good state of repair.

8.2 Nomenclature of Expansion Joints

The nomenclature for the components of expansion joint systems varies among the many users and so a brief discussion describing the various terms is provided.

- Joint/Gap—the distance between the deck or superstructure ends which allows for joint movement. It is important to note that this distance changes with temperature.
- Backer Rod—a foam-like material commonly used to span the gap and prevent liquid sealers from falling through the space. Typically shaped as a long cylinder it is compressed into the joint. Used in pourable joints and asphaltic plug joints.
- Bridging/Gap plate—a plate covering the gap beneath an asphaltic plug. It is used to keep the plug seal from moving into the joint.
- Asphaltic Plug/Plug—a flexible material that forms the road surface atop the joint and seals the joint. The material is a specially formulated asphaltic binder mixed with

aggregate to withstand cracking at low temperatures and tracking/softening under wheel loads at high temperatures. Typically used with Asphalt Overlays.

- Header/Nosing—the material bonded to the deck/approach slab which resists the wheel loads prior to the gap, and provides a vertical surface for the attachment of the expansion joint system. Sometimes made of hydraulic cement concrete and sometimes made of elastomeric concrete.
- Pourable Seal/Silicone Seal—a flowable material placed directly in the gap upon a backer rod which adheres to the vertical faces of the header material to seal the expansion joint. More recently used with elastomeric headers. The Silicone Seal is the most commonly used pourable seal today.
- Compression seal—a preformed joint seal squeezed and inserted into the gap using lubricants which also serve as adhesives that bond the seal to the header material. There are open cell compression seals which are typically preformed neoprene semi-hollow rectangular shapes and closed cell compression seals with solid rectangular shapes with a foam-like appearance. The joint systems must remain in compression to be water tight.
- Inflatable Seal—a preformed neoprene open cell seal which is inserted into the gap and air pressurized. The seal profile and epoxy adhesive provide a tight bond to the header (typically elastomeric concrete) during the inflation process. This is a sole source seal.
- Extrusions/Armor Plate/Metal Runner—a steel material anchored to a header/nosing joint face to which the expansion joint seal is attached.
- Anchorage—steel studs welded to armor plates to anchor the plate in the header.
- Strip/Gland/Membrane Seal—a preformed neoprene V-shaped seal that is rigidly attached to an armor plate with a rail profile. The seal will be water tight in tension or compression.
- Cushion/Plank Seal—a steel reinforced neoprene pad recessed into the deck over the gap rigidly attached on both sides of the seal. The attachment consists of rods anchored in the deck and threaded on top with nuts holding the pad down.
- Modular/Modular Dam—an expansion joint system consisting of a series of rail profiles (called separator beams) which are rigidly connected to a series of neoprene shapes and are supported longitudinally by support bars.
- Separator Beams—beams that support seals in a series in a modular joint. The edge beam is the first in the series and attaches to the deck end usually an armor plate with a rail profile holding one side of a strip/block seal. The center beams are the beams in the series that do not attach to the deck. They are steel extrusions with a rail profile on two sides which provide attachment to strip/block seals.
- Support Bar—supports the separator beams of a modular expansion joint in the longitudinal direction.
- Finger/Tooth/Cantilever Joint—an open joint with interlocking steel fingers/teeth.

8.3 Common Types of Expansion Joints

8.3.1 Compression Seal

A compression seal is a commonly used joint system that relies on compression to maintain water-tightness. As described in Section 8.2, the seal is inserted into the joint using lubricants which may also serve as adhesives. There are two types of compression seals; open cell and closed cell foam.

Figure 8.1 illustrates an open cell seal. The open cell is made of a neoprene elastomeric material with internal webbing, which allows it to compress yet gives it a stable interface with the deck end. The deck end may be armored or may contain a header to which the seal is attached. Typically a stop bar or block out is provided directly beneath the seal to keep it in place. Movements up to 2-1/2 inches or more can be accommodated with this type of open cell seal. Because compression seals work in compression, sizing the seals becomes particularly important when constructing or replacing these seals in warmer temperatures. The seals must be large enough for the joint opening required for the coldest temperatures. Guidance regarding the placement of these seals is provided in Section 8.6.2.1.

Closed cell foam compression seals are another type of compression seal that agencies use. The seal is typically a preformed low density ethylene vinyl acetate polyethylene (foam) material that is bonded into place with a two component epoxy adhesive. The deck ends may be armored or may contain a header to which the seal is attached. Because of the epoxy bonding, closed cell foam seals do not rely solely on compression to prevent water and materials from flowing through the joint. These foam seal typically have a working range of movement of about 60 percent compression and 30 percent tension. Movements of up to 4 inches can be accommodated by some closed cell foam seals. Sizing the seals becomes particularly important when constructing or replacing these seals in warmer temperatures. The seals must be sized and installed properly for the joint opening required at both the warmest and coldest temperatures. Other important considerations when placing both types of compression seals include ensuring a uniform joint width and ensuring that the joint seal faces are vertical with no spalls. It is also important that the seals are set at the proper distance below the deck.

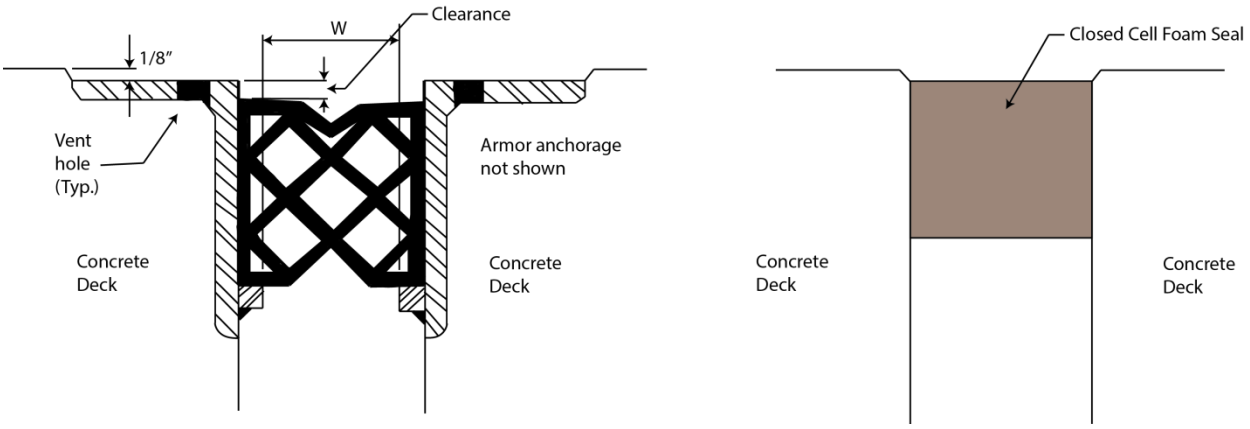


Figure 8.1 Compression Seals. Open Type (left). Closed Cell Foam Type (right).

8.3.2 Pourable Joint Seals

Pourable joint seals have been used for many years for small movement joints. Figure 8.2 shows a schematic detail of an older version of this joint type (generally used for small movements up to 1 inch). A pre-molded backing material is placed below the sealant to keep the sealing compound from falling through the joint. Early sealers were heated asphalt or coal tar products; these sealers tended to split, debond, and become damaged by non-compressible debris lodged into the joint. They were also damaged as a result of deck edge damage.

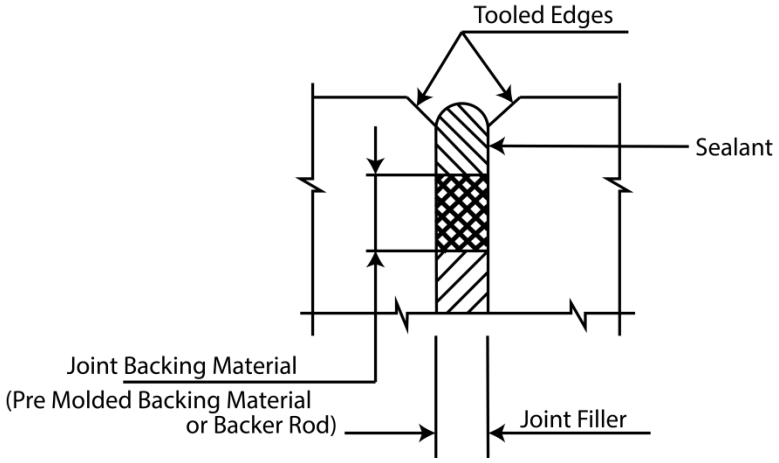


Figure 8.2 Poured Joint Seal

Newer pourable seal joints are shown schematically in Figure 8.3 (they can be used for movements up to 3 inches). The sealer is made of a self-leveling, rapidly curing silicone material. The headers are typically elastomeric concrete, which reinforces the deck ends and is more durable material. The poured shape is advantageous if the joint walls are not perfectly parallel or vertical, particularly when bonded to the elastomeric header. This type of joint is also relatively easy to repair, to remove a section, to clean the vertical walls, and to pour a new section into place with little traffic interruption or work zone hazards.

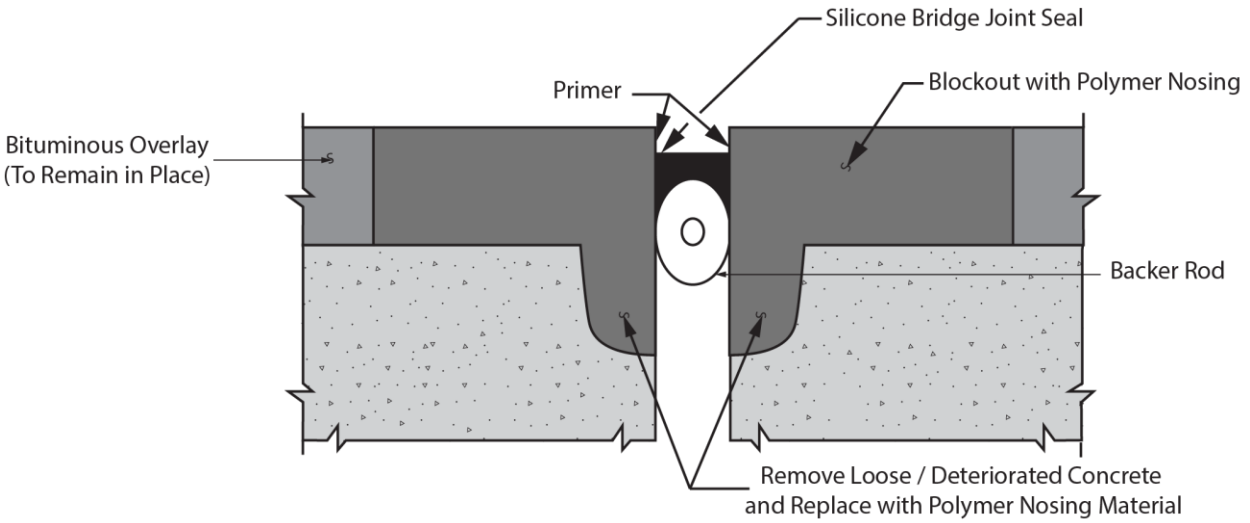


Figure 8.3 Newer Pourable Joint Seal

It is important that the silicone does not bond to the backer rod and that the sealer is thinner in the middle than at the deck end. This is because as the joint opens, the seal is forced to stretch within its thinnest cross-section at its middle to accommodate the movement rather than stretching at its ends and possibly breaking its bond with the deck or header resulting in future joint leakage. The sealant material should be applied along the existing concrete joint faces or applied to a clean and sound header and placed slightly below the riding surface (1/2 inch). If the silicone sealer is applied along the existing concrete joint faces, the existing concrete should be cleaned using abrasive blast cleaning. The silicone should be placed at the mid-portion of the temperature range it is likely to experience (or mid-portion of the expected joint opening range) but above 40 °F.

8.3.3 Asphaltic Plug Seal

An asphaltic plug seal is a flexible material that forms the road surface atop the joint. It is used for small movement joints (up to 1.5 inches). The material is a specially formulated asphaltic binder mixed with aggregate to withstand cracking at low temperatures and tracking/softening under wheel loads at high temperatures. It is typically used with asphalt overlays. Figure 8.4 provides a schematic of this seal type, and a detail is provided in Figure 8.5. The sealer shown in Figure 8.5 is typically a silicone or polymer bridge joint sealer. A series of locating pins that keep the bridging plate in position during placement of the asphaltic plug material is also shown.

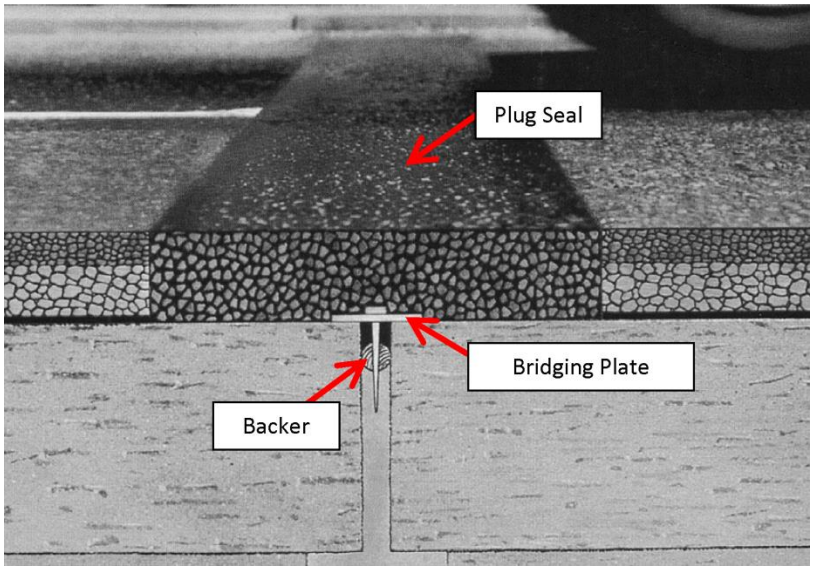


Figure 8.4 Asphaltic Plug Seal

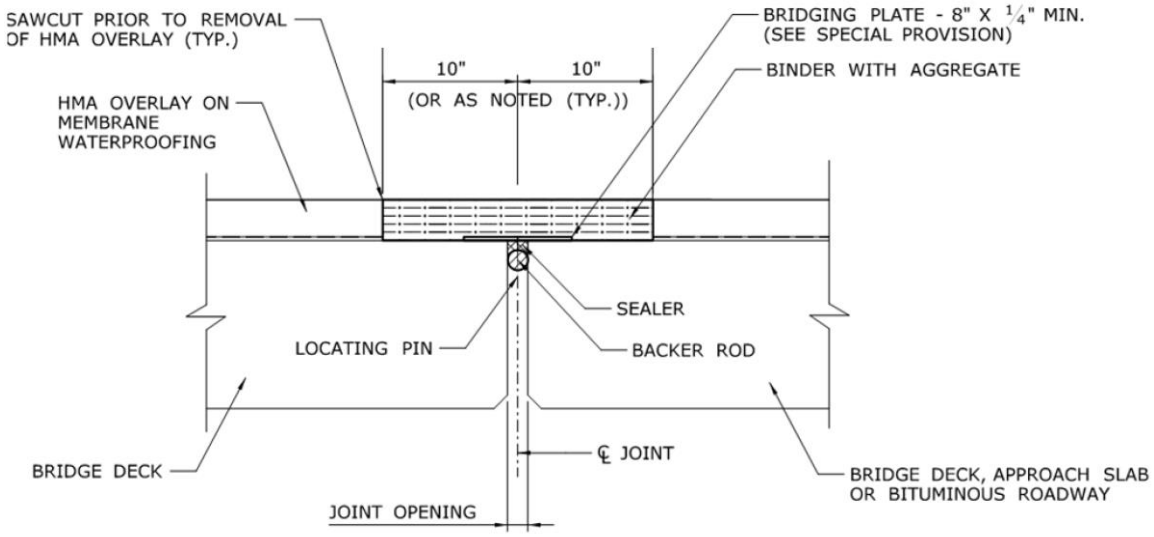


Figure 8.5 Detail for Asphaltic Plug Joint

The asphaltic plug seal offers many advantages including ease of installation, low cost of installation and repair, and a smooth riding surface with a low-frequency of snow plow damage.

The disadvantages lie in the fact that the system doesn't work in vertical applications, so alternate seals need to be provided at curbs and parapets. Additionally, problems noted on some installations include softening in hot weather and cracking in cold weather, which may cause debonding at the seal/pavement interface. This joint is not recommended in high volumes of stopping traffic or in skews over 45 degrees. One manufacturer suggests using this joint on secondary roads.

Some agencies are conducting tests for the use of elastomeric concrete in plug joint systems. This type of joint system would be similar to the asphalt plug joint except the specially formulated asphaltic binder mixed with aggregate would be replaced with elastomeric concrete.

8.3.4 Strip Seal

The strip seal expansion joint system is characterized by a preformed v-shaped neoprene seal which is rigidly attached to an armor plate with a rail profile. The armor plates are anchored to the deck. The seal will be water tight in tension or compression. A schematic detail is shown in Figure 8.6. This joint type can accommodate up to 4 inches of movement.

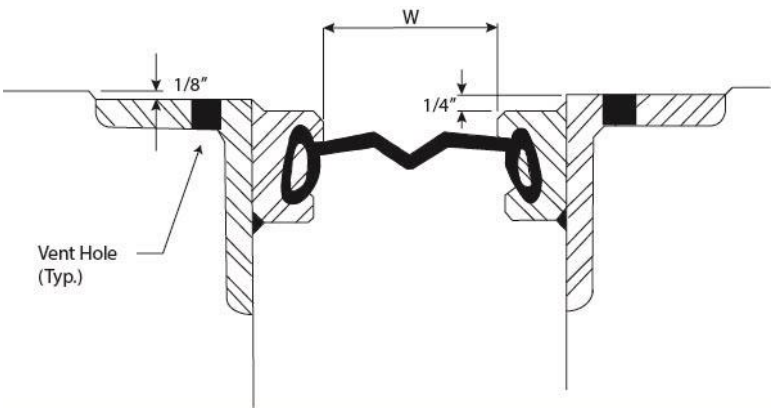


Figure 8.6 Strip Seal

While the strip seal is more durable than other seals, debris accumulation creates problems for this type of joint since its exposed surface area can be larger than other seal types. Non-compressible material accumulates in the joint crevices as the joint expands. Rupture of the seal then occurs as it closes in warmer temperatures. Traffic impact over debris filled joints can also wear down the membrane. This can result in the loss of water-tightness which may not be easy to see when looking at the seal. There are also problems along vertical and horizontal changes in cross-section such as along curb lines. Replacing the seal can also be difficult. Skews can also be problematic particularly when the skew angles are similar to plow blades. Dislodged and damaged armor plates can create traffic safety problems as well.

When installing strip seals in existing concrete decks, it is often required to field adjust the anchorage to avoid conflicts with deck reinforcement. An example of problems that can develop when installing a strip seal is shown in the sequence of Figure 8.7 through Figure 8.9. Figure 8.7 shows conflicts between shear studs and K bars in diaphragms. In Figure 8.8, the contractor initially bent shear studs to avoid conflicts with existing steel. Angles with bent shear studs were rejected. With congested steel, finesse is needed to put angles into place and not force them down. To help secure the rail in place until the concrete hardens, an angle support as shown in Figure 8.9 can be used. Also shown in Figure 8.9 is a vertical installation of the armor plate near the curb. This type design detail capability can allow the vertical installation of the strip seal so the joint can be completely closed at the curb, fascia or at a barrier wall.

Figure 8.7 Conflicts with Bottom Shear Studs and K bars in Diaphragms

Figure 8.8 Bent Shear Studs

Figure 8.9 Angle Support Bolted to Metalized Angles for Support Until Concrete Hardens

8.3.5 Modular Expansion Joint

Modular Joints are needed to accommodate large joint movements openings, e.g., those over 4 inches and as large as 24 inches. The modular expansion joint system is made up of 3 major components including:

1. Seals
2. Separator beams (edge and center beams)
3. Support bars

A typical schematic is shown in Figure 8.10. The seals and beams form the roadway riding surface. The edge and center beams are steel shapes often referred to as rail profiles to which the seals are attached. The separator beams are supported on support bars which are spaced at frequent intervals. Modular joints can handle movement in the longitudinal, transverse, and vertical direction, as well as vertical rotations. Bearings and springs are also used to accommodate movement. A photograph of a modular expansion joint installation is shown in Figure 8.11.

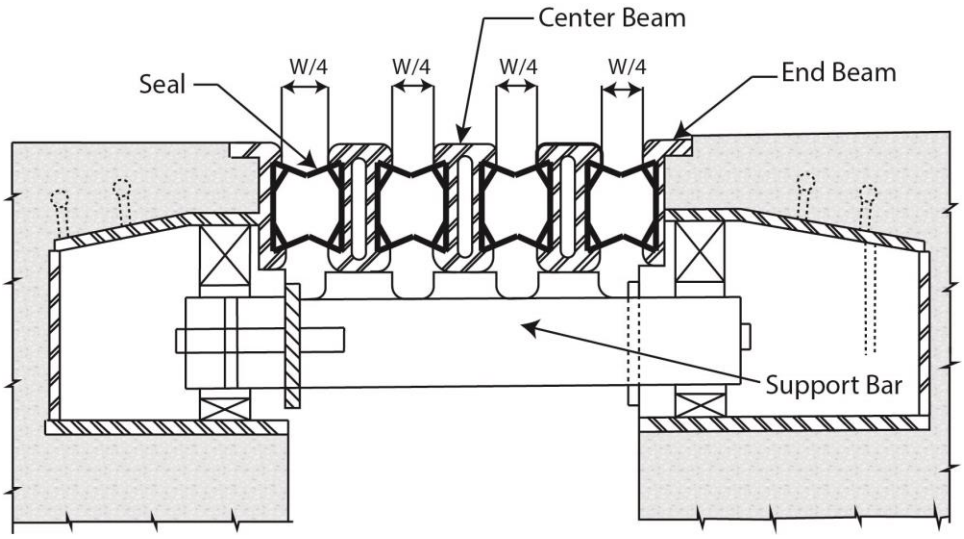


Figure 8.10 Modular Expansion Joint

Figure 8.11 Installation of Modular Expansion Joint

It is obvious that there are many parts to this joint system, and as such it cannot be considered maintenance friendly. These joints are subjected to millions of high dynamic stress cycles due to passing vehicle loads, so the framing system must be designed for large fatigue stresses. The seals must also be very durable. When welds crack, equalizing springs are damaged, and seals pull-out or rupture; these pieces are difficult to repair. The initial install is also difficult, including setting to the proper line and grade and joint opening; ensuring the proper block out reinforcement and concrete consolidation; and proving field splices when necessary for stage construction; these installation difficulties add to the concern of future maintenance. Although these joint types are fraught with inherent complexities, engineering improvements are continuing to be made, particularly in the realm of fatigue resistant detailing. There are not many joint types available for long continuous bridges which can handle the anticipated movement. As such, we can expect there will be more modular expansion joints as we build new bridges with fewer traditional expansion joints.

8.3.6 Finger Expansion Joints

Finger joints are typically classified as open joints. Most agencies resist using open joints, but finger joints offer the ability to handle large movements (greater than 4 inches) and are often

an alternative to modular joints. A schematic of a finger joint is shown in Figure 8.12, and a photograph is shown in Figure 8.13. Because this is an open joint, some agencies install a drainage trough to capture water and debris which protects the bridge elements below the joint. The slope of the trough system should be steep enough to ensure adequate drainage. Easy accessibility is also required. Alternatively, closed drainage systems can be installed if the joint is placed behind the backwall (this concept will be discussed in Section 8.7.1). The maintenance of the drainage trough is critical and is considered a disadvantage of this joint type.

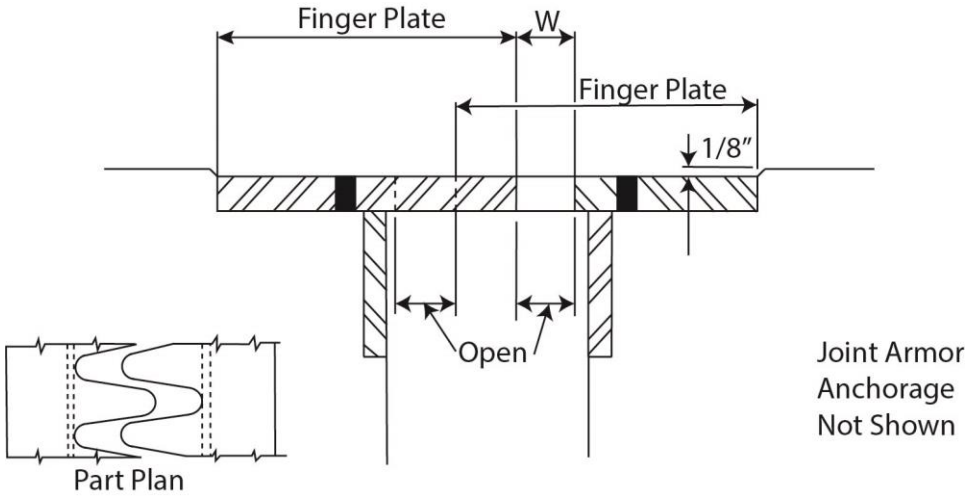


Figure 8.12 Cross-Section of a Finger Joint

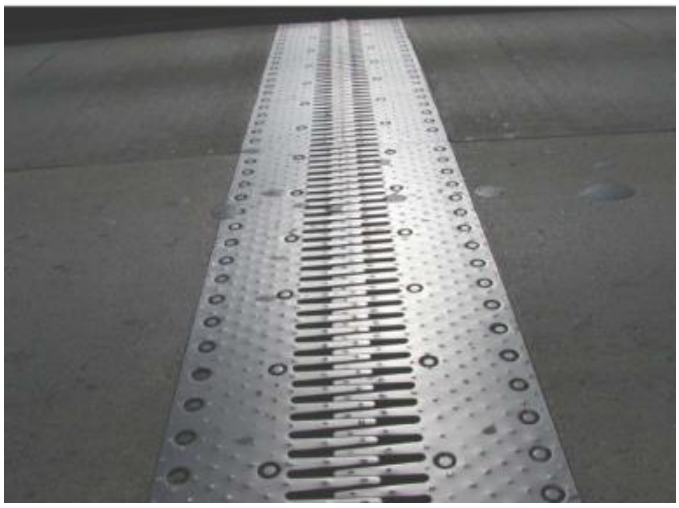


Figure 8.13 Photo of a Finger Joint

Other maintenance problems associated with finger joints include: misalignment (vertical and horizontal direction), broken finger plates, broken securing bolts, and concrete anchors. Although an open joint, debris can still become lodged in the gap, thus preventing the longitudinal movement. This scenario imposes undesired forces on the bridge and may cause damage to the teeth. Finger joints require relatively flat grades for proper installation.

Despite the potential issues of finger joints, this joint is easier to maintain than its large movement counterpart (the modular joint) with fewer moving pieces.

8.4 Expansion Joint Defects

Typical joint disadvantages have been described in the previous section for common joint types. In this section, the methodology for evaluation of common defects and understanding their significance and severity is described.

Although many of the joint defects are described here in visible and measurable ways according to the function of their components, it is important to remember that an expansion joint's performance is compromised when it is failing to do what it is meant to do: withstand traffic loads and allow bridge movement, provide a smooth riding surface and safe surface for pedestrians and bicyclists (where they are allowed), provide some skid resistance, minimize noise and vibration (which are defect indicators), and prevent water from infiltrating and damaging other bridge components.

8.4.1 Leakage

Joint leakage is the clearest indicator that the joint system is not functioning properly. Joint leakage is best determined from the joint underside. It may be difficult to determine joint leakage if the weather has been dry for a period before the inspection. Deterioration of the components beneath the bridge is the best indicator that there is water leakage. What to look for:

What To Look For

- Water staining, ponding water on the abutment shelf, pier cap or abutment backwall—minor deterioration
- Water leakage is obvious and there is damage to protective systems—moderate deterioration
- Water is passing through the joint freely and there is damage to bearings (Figure 8.14), abutment shelf/pier cap, abutment backwall or girder ends (Figure 8.15) and end diaphragms (Figure 8.16)—major deterioration

Figure 8.14 Damage to Bearings – Major Deterioration

Figure 8.15 Damage to Steel Beam Ends – Major Deterioration

Figure 8.16 Damage to Concrete End Diaphragms – Major Deterioration

8.4.2 Condition of Road Surface Adjacent to Joint

The condition of the road surface adjacent to the joint can deteriorate to the point that the joint system will also fail. What to look for:

What To Look For

- Transverse cracking along the roadway that is narrow and shallow, riding surface is good—minor deterioration
- Transverse cracks are deeper (not yet the full depth of the pavement structure) and pot holes are occurring or likely to occur, headers are exposed—moderate deterioration
- Longitudinal and transverse cracking along the roadway that is narrow and shallow, riding surface is good—minor deterioration
- Longitudinal and transverse cracks that are deeper (not yet the full depth of the pavement structure) and pot holes are occurring or likely to occur, headers are exposed—moderate deterioration
- Partial/full depth deterioration of the roadway surface, pot holes exist adjacent to the joint, the headers and/or armor plates are exposed—major deterioration

8.4.3 Condition of Seal

The condition of the seal is a primary indicator of the degree of leakage to expect. What to look for:

What To Look For

- The seal has some cracks, minor tears and some breaches and only a small amount of debris is evident—minor deterioration.
- The seal is missing or substantially breached at some point, with a significant amount of debris—moderate deterioration.
- The seal is missing, significantly pulled out and filled with debris—major deterioration.

8.4.4 Asphaltic Plug Joint Defects

8.4.4.1 Wearing Surface to Plug Interface

Like other joint systems, the wearing surface to the asphaltic plug interface should be well bonded to prevent water infiltration. What to look for:

What To Look For

- The plug/wearing surface has debonded with narrow transverse cracks which are relatively shallow (less than an inch)—minor deterioration.
- The plug/wearing surface shows wider transverse cracks which are also deeper (although not the full depth of the wearing surface) and the adjacent wearing surface is showing some signs of distress—moderate deterioration.
- The plug/wearing surface is debonded the full depth of the interface and both the plug and the wearing surface are breaking up from the lack of support, leakage is evident—major deterioration.

8.4.4.2 Plug Seal

The condition of the plug seal is a primary indicator of its effectiveness. The seal can break up and lose material in cold weather or shove/ track in hot weather; both will compromise water tightness. What to look for:

What To Look For

Plug Material Loss

- Small, shallow (less than 1 inch) potholes in the plug—minor deterioration.
- Larger, deeper (2 inches) potholes exist in the plug, and there is noise as the driver traverses the joint—moderate deterioration.
- Potholes to the full /partial depth of the joint, plug material is breaking up —major deterioration.

Plug Material Shoving/Tracking

- Small depressions and limited material tracks outside the joint, purely aesthetic and doesn't appear to affect the water-tightness of the joint—minor deterioration.
- Depressions and tracked material are evident, small mounds of binder can be seen along the curb line, and rideablity has been affected—moderate deterioration.
- The plug material has been displaced to the point that the plug is no longer effective—major deterioration.

8.4.5 Condition of Header Material

The condition of the header can deteriorate to the point that the joint will become ineffective. What to look for:

What To Look For

- Cracking in the header is minor and is not affecting the joint effectiveness—minor deterioration
- Cracks in the header are fairly extensive and are compromising the header interface with the road surface and the interface with the joint—moderate deterioration
- Header material is breaking up to the extent that other elements of the joint are compromised and leakage is the result—major deterioration

8.4.6 Condition of the Armor Plates and Steel Rails

The condition of the armor plates and rail elements may deteriorate to the point that the joint will not function properly. Failure of these components can become a serious traffic hazard.

What To Look For

- Slight deformities in the rails exist but there is no vertical deflection—minor deterioration
- Crack or tear < 20% of the rail width; or deformation with no vertical deflection—moderate deterioration
- Crack or tear > 50% of the rail width; or a deformation creating vertical deflection—major deterioration
- Loose or missing hardware anchoring armor plates/angle to slab
- Loose or vibrating components under traffic

Recognizing defects in joint systems can be particularly important in applying Preventive Maintenance. Bridge Joints that are in perfect condition and those that are failed are the easiest to distinguish. It is important to recognize joints with minor to moderate deterioration to apply Preventive Maintenance or repairs before the leakage is completely unrestrained.

8.5 Preventive and Basic Maintenance of Deck Expansion Joints

In this section, Preventive Maintenance refers to the on-going scheduled activities aimed at extending/maintaining the service life of bridge deck expansion joints. Scheduled maintenance activities become important to keeping the expansion joints performing properly. Most small movement joints do not have long service lives (average 5 years) and they are typically only inspected every two years. The amount of detail regarding expansion joint defects will vary from inspection to inspection and within various agencies. This makes annual maintenance activity a necessity. If maintenance is proactive and done at an appropriate frequency and time of year much can be gained.

8.5.1 Washing to Remove Debris

8.5.1.1 Bearing Seats

Although not a part of the expansion joint, the bearing seats should be cleaned and all debris removed and disposed. Debris should be manually removed by brushing, chipping and scraping. Then the bearing seat should be flushed with pressurized water to remove chemicals, deicing salts and debris that cannot be removed manually. This is a step that allows the bridge maintainer to see if damage is occurring as the result of leaking joints and prevent further damage. The bearing seat drainage should be checked and cleaned as well.

It is important when flushing is performed in the vicinity of water bodies to limit the debris entering the water body. Use sedimentation control devices to limit stream bank erosion.

Cleaning of the bearing seats should be performed annually after the snow season.

8.5.1.2 Cover Plates at Parapets and Curbs

Cover plates should be removed and the joint should be cleaned and debris removed. There may also be vegetation in this area which should also be removed.

8.5.1.3 Compression Seals, Strip Seals, and Silicone Seals

These seals should be high-pressure jet washed to remove all debris annually (higher frequencies should be considered for high volume roads). This should be done in the early spring after the snow season.

8.5.1.4 Modular Joints

The seal should be cleaned as described above. The systems below the seal should also be cleaned annually; this service may need to be supplied by the manufacturer.

8.5.1.5 Finger Joints

These joints should be high pressure jet washed including all components above and below the road surface to remove debris. Because debris not only affects the condition of the joint and the bridge components directly beneath, but can restrict movement of the bridge imposing undesired forces on the bridge, it is recommended that the washing be semi-annual in the fall and early spring. An example of a finger joint trough assembly is shown in Figure 8.17.

Figure 8.17 Finger Joint Trough Assembly

8.5.2 Cleaning out Drainage Systems

All troughs should be cleaned and flushed, in addition to pressure washing other cleaning tools such as rods and shovels should be used to ensure that all debris is removed including any deposits outside the trough. The troughs should be examined for soundness and be replaced if torn or punctured. Troughs should hang freely. The slope should be self-draining; a 5 percent grade is recommended. The cleaning of the troughs should be performed annually in the spring after the snow season.

8.5.3 Replacing Seals

In this section the replacing of seals relates to small movement joints where there is no seal or a pourable joint seal is missing or ineffective. A suggested procedure for replacing hot or cold poured seals is shown below.

Suggested Procedure

Pourable Seal Installation

1. Remove old sealants and debris from joint.
2. Sandblast the joint surfaces to remove any remaining joint sealant residue and roughen the surfaces to provide a surface to which the sealant will bond.
3. Inspect the joint surfaces to assure complete removal of prior joint material and that the surfaces are sound and undamaged.
4. Apply primer to the joint surfaces in accordance with the sealer manufacturer's recommendations.
5. Install backer rod.
6. Pour sealant over the backer rod between the joint surfaces to the elevations required.
7. Allow sealant to cure in accordance with the manufacturer's recommendations prior to opening to traffic.

Replacing entire seals and joint systems will be elaborated upon in Section 8.6.

8.5.3.1 Hot Poured Sealant

Applying hot poured sealants should be performed at temperatures between 45 °F to 80 °F with no significant precipitation anticipated for a 24 hour period. Remove the remnants of the old seal and any debris to at least three times the width of the joint. Repair the joint as necessary. Ensure that the joint edge surfaces are dry. Install a backer rod to an inch below the roadway surface. Clean out the area with an air compressor and apply the sealant.

8.5.3.2 Cold Poured Sealant

Applying a cold poured silicone sealant should be performed at temperatures between 55 °F to 80 °F. There should be no anticipated precipitation for 36 hours. This repair should only be performed on joints with adjacent deck surfaces that are clean and sound. Old sealants and debris should be removed to 3 times the width of the joint. The joint surfaces should be sand-blasted and inspected to assure that no residue remains. Joint surfaces should be primed per manufactures recommendation. Install the backer rod. Install the silicone sealant in accordance with Figure 8.18. This Figure shows both correct and incorrect silicone sealer installation. It is noted the silicone sealer should be concave (filler convex upward) so that its cross section is the thinnest at its center. The purpose is to facilitate stretching of the sealer within a thin cross section at its center to thereby avoid creating stretching and/or larger forces at the sealer ends that could debond the sealer from the joint wall. Examples of before and after photographs of a silicone sealant installation are presented in Figure 8.19.

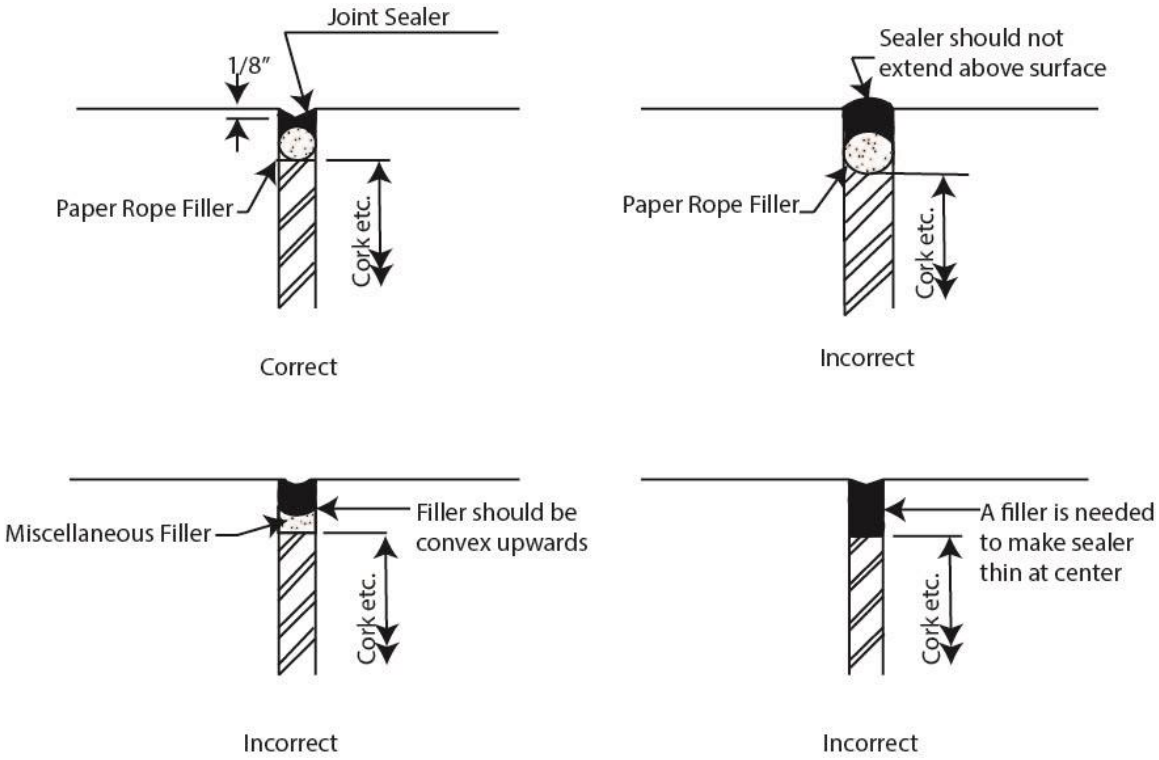


Figure 8.18 Silicone Sealant Installation

Figure 8.19 Before (left) and After (right) Photos of Silicone Sealant Installation

8.5.4 Sealing Headers

Concrete sealants are very effective in reducing concrete deterioration. Sealants must be applied properly to be effective. Concrete sealants are discussed in detail in Chapter 7. Consideration should be given to sealing concrete headers.

8.6 Repair and Rehabilitation of Deck Expansion Joints

A bridge maintenance crew will often be called upon to repair and rehabilitate bridge deck joints. This will go beyond simply replacing the seals of otherwise intact joints systems.

8.6.1 Replacing Joints with Damaged Headers

8.6.1.1 Replacing Armored Joint with New Concrete Headers

A suggested procedure for replacing armored joint with new concrete headers is shown below.

Suggested Procedure

Replacing Armored Joint with New Concrete Headers

1. Remove the armor plates, anchors, and deteriorated concrete.
2. The extent of the concrete to be removed should be marked out and saw cut approximately 1 inch deep. The concrete should be removed by hand tools and small pneumatic hammers (30 pounds above the reinforcing steel, and 15 pounds below the reinforcing steel).
3. Set and secure the joint edge form.
4. Clean the area by abrasive blast cleaning.
5. Place required new reinforcing steel and remove/replace/lap existing corroded rebar.
6. Take care to ensure that the concrete surface of the header matches the deck grade. Setting the joint too high or too low will create undesirable vehicle loads.

8.6.1.2 Replacing Headers with Polymer Concrete (Elastomeric Concrete)

A suggested procedure for replacing headers with polymer concrete (elastomeric concrete) is shown below.

Suggested Procedure

Replacing Headers with Polymer Concrete (Elastomeric Concrete)

1. Remove the armor plates and anchors (if they are present) and also deteriorated concrete.
2. The extent of the concrete to be removed should be marked out and saw cut approximately 1 inch deep. The concrete should be removed by hand tools and small pneumatic hammers (30 pounds above the reinforcing steel and 15 pounds below the reinforcing steel).
3. On occasion the surface will be an asphalt overlay; this surface shall be saw cut as well, and the asphalt removed.
4. Ensure the block out is cleaned of old joint seals, concrete debris, asphalt overlay, water proofing membrane, dirt, grease, and any other deleterious material.
5. Form the joint edge to achieve the proper opening. This is based upon the seal type, the span length, and the temperature.
6. The substrate to which the elastomeric concrete is to be placed against should be coated with a primer.
7. Place polymer concrete header in accordance with manufacturer recommendations. The header must be flush with the roadway surface and should be cured in accordance with manufacturer recommendations.
8. A number of seals can be used with this header type including compression seals, silicone seals, and strip seals.

8.6.2 Replacing Seals

8.6.2.1 Compression Seals

Compression seals offer the advantage of not requiring armoring or anchoring to their headers. The difficulty in replacing a compression seal lies in sizing the seal to be in compression when at its most contracted state.

Existing seals and other debris should be removed from the joint opening. The joint edges should be sand blasted and cleaned. Elastomeric or concrete headers are both suitable. The seal should be coated with an adhesive which will serve as a lubricant until the material becomes tacky. The seal size should be determined prior to placing the seal. It is best to install these seals in cooler temperatures. An easy to use Joint Calculator was developed by the New York State Department of Transportation for closed cell seals; it calculates the total joint movement expected based upon AASHTO guidelines for the temperature ranges for a given region. It then calculates the joint opening at three temperatures (high, low and 68 °F). The width and depth of a closed cell foam material can then by calculated using inputs that can be measured or known on the day of installation (temperature of the primary member, length of primary member contributing to the total movement, joint opening at the time, skew angle of the bridge, and the maximum compression limit percentage of the joint material). This type of

spreadsheet can be used by any locality for closed cell compression seals. Figure 8.20 shows the inputs and outputs of the Expansion Joint Calculator.

The Expansion Joint Calculator spreadsheet has simple formulas to calculate the joint movements at various temperatures. All the green boxes are inputs that are known; either about the bridge or the seal characteristics provided by the manufacturer. The tan boxes contain the formulas relevant to calculating movements based upon the length of the span and the coefficient of thermal expansion and the degree of joint compression based upon the thermal expansion. This can be easily replicated for other localities.

NYSDOT Expansion Joint Calculator

Make selections for the following categories:

NYSDOT Region	R7	
Primary Member Type	Steel	

Input values for the following categories:

	Exist Temperature of Primary Member	55	°F
*	Length of Member(s) contributing to total movement @ joint	124	Feet
*	Exist Joint Opening (perpendicular to header) ⊥	2	inches
*	Skew Angle ($0^{\circ} \leq \theta < 90^{\circ}$)	18	Degree Skew
	Max. Compression limit % of joint material (from Manufacturer)	60	%

Results: (Perpendicular to header)

Joint Opening at 68°F (⊥)	1.88	inches
Joint Opening at Coldest °F (⊥)	2.69	inches
Joint Opening at Warmest °F (⊥)	1.54	inches
Total movement of Joint within full Temp range (⊥)	1.15	inches
Width of New Closed Cell Foam (use Coldest Temp width (⊥))	**2.69**	inches
Depth of New Closed Cell Foam (if Manuf. info. not available)	**2.50**	inches
Acceptablility based on Manufacturer's Compression limit	**OK**	
Compression @ Warmest Temp	42.75	%
Compression @ Exist Temp	25.65	%

Results: (Parallel to primary member)

Joint Opening at 68°F (//)	1.98	inches
Joint Opening at Coldest °F (//)	2.83	inches
Joint Opening at Warmest °F (//)	1.62	inches
Total movement of Joint within full Temp range (//)	1.21	inches

Procedure B Temperature Ranges - AASHTO

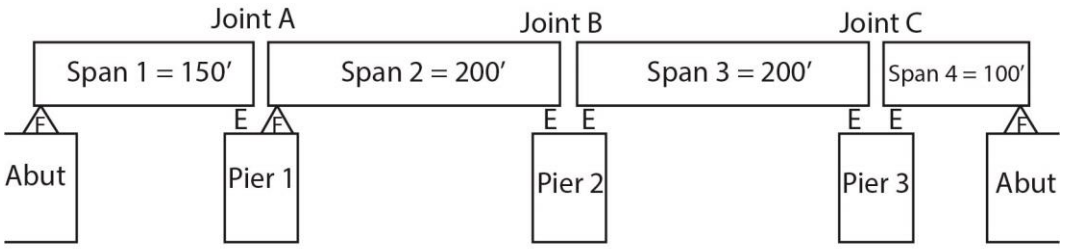
Coldest Temp for Primary Member used in your Region	-20	°F
Warmest Temp for Primary Member used in your Region	+105	°F

* See Diagram for clarification

Expansion & Contraction relate to Primary Member
Tension & Compression relate to Joint Material

Figure 8.20 NYSDOT Expansion Joint Calculator

Figure 8.21 and Figure 8.22 are used to help field personnel determine the inputs to the expansion joint calculator in Figure 8.20. Note that Joint A = 150 LF since Span 2 bearing is fixed, therefore no movement and since Span 1 bearing is expansion, the full movement of Span 1 (150 LF) contributes to the movement of the joint.



Length of members contributing to movement of Joint:
Joint A = 150 LF
Joint B = 300 LF
Joint C = 200 LF
F = Fixed Bearing
E = Expansion Bearing
Assume all bearings are functional

Figure 8.21 Length of Member Contributing to Joint Opening

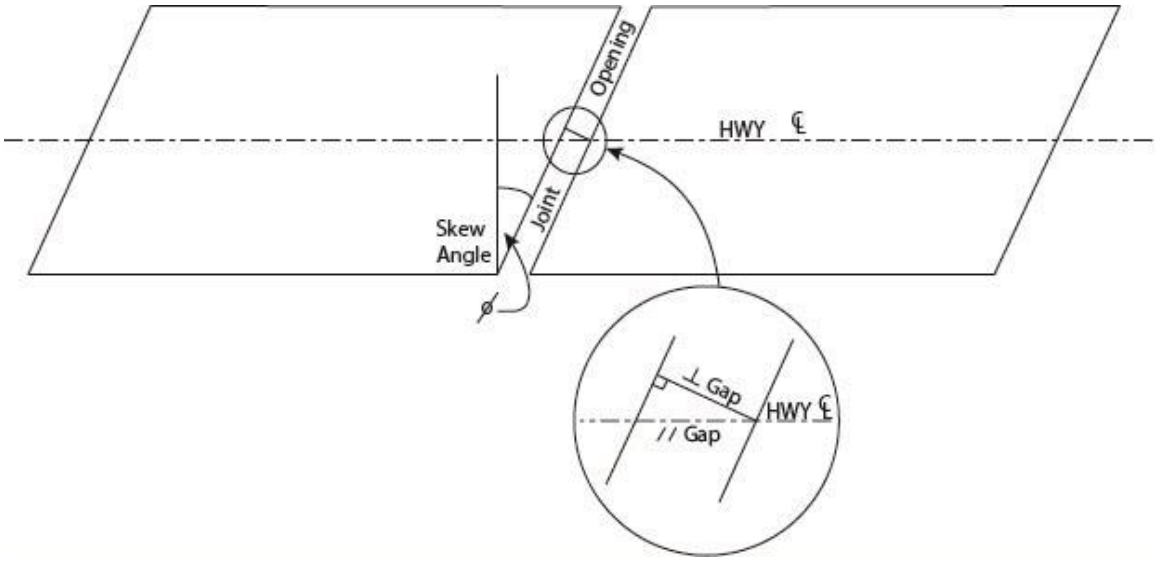


Figure 8.22 Joint Opening and Skew Angle

Compression seals can also be modified to turn up a curb or parapet. Figure 8.23 shows how holes can be drilled into the seal and cuts made to the holes to provide upturns, and 90° wedges can be cut to make 90 degree downturns. The seal installation at the curbs and parapets follows a similar procedure to the deck joint opening.

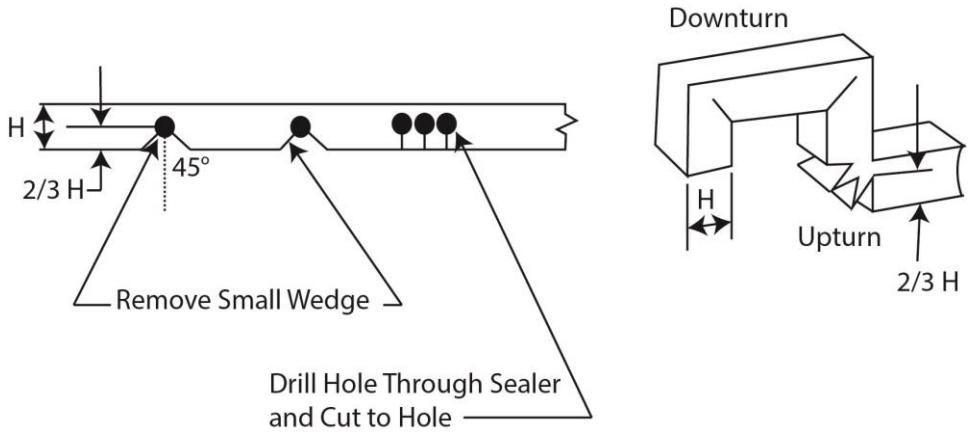


Figure 8.23 Schematic of Upturn/Downturn

8.6.2.2 Asphaltic Plug Joints (APJ)

Asphaltic Plug Joints are easily installed and a low cost bridge expansion joint type. They can replace joints used with an asphalt wearing surface. Although they are easy to install, careful attention to the placement of the plug is required to obtain its desired performance characteristics (Figure 8.24).

Figure 8.24 Asphaltic Plug Joint with Binder Placed Atop Backer Rod Prior to Bridging Plate Installation

A block out for the plug joint should be provided. The overlay should be saw cut at the block out limits. It is desirable to saw cut freshly laid asphalt, this will require that the existing APJ be milled out and temporarily paved over. The block outs are typically 20 inches wide. The depth of the saw cut should be established by drilling holes to determine the pavement depth. The existing pavement is then removed with jackhammers and hand tools. Care should be taken to ensure the existing deck is not damaged during the process. Once exposed the concrete deck should be repaired and should be level to ensure the levelness of the bridging plate, it should not rock or deflect. The horizontal and vertical faces of the block out should be abrasive blast cleaned, and dried. The backer rod should then be installed 1 inch below the top of the deck surface. Once the backer rod has been installed the asphalt binder is placed above the backer

rod and applied to all surfaces of the block out. Bridging plates are then installed; the plates should be centered over the joint and secured with locating pins along the centerline. The joint is then tanked (flooded) again with the binder material.

The aggregates and binder are prepared for mixing. The aggregate should be heated and then placed in a separate drum to be coated with the asphalt binder (this method ensures best results for coating the aggregate). The coated aggregate should then be loosely placed in the cut out, then leveled and compacted in 2 inch lifts. The aggregate mixture is then tanked (flooded) with the binder material to fill the voids, only the tips of the aggregate should be seen. The next lift is placed in a similar manner. In the final lift the aggregate should not protrude above the wearing course surface. The binder is applied to the final lift ensuring there are no voids and no stones protruding (Figure 8.25). The final course is topped with a thin anti-tracking layer to provide skid resistance.

Figure 8.25 Plug Installation Showing Coated Aggregate and Binder

The temperature at the time of installation, as well as the temperature of the binder and the aggregate, is important to the installation of this joint. It is critical to know the temperatures of the materials and to calibrate any temperature monitors. The cleanliness of the block out and the aggregate are also important to monitor.

A suggested procedure for asphaltic plug seal installation:

Suggested Procedure

Asphaltic Plug Seal Installation

1. Saw-cut and remove the wearing surface and waterproofing membrane (if applicable) from the deck.
2. The block out is then cleaned and dried, repairing concrete as necessary.
3. The backer rod is then placed allowing at least 1 inch depth of binder material from the top of the deck. Note: the binder material is typically polymer modified asphalt that is commercially available for asphalt plug seal installation.
4. Then the joint is tanked (flooded) with binder material heated to manufacturer specifications.
5. Next the bridging plate is installed (Figure 8.26), centered over the gap with locating pins, and the binder is placed atop the bridging plate.
6. The aggregate must be clean and graded properly then heated to the appropriate temperature. The heated aggregate is then mixed with heated binder to provide a coating.
7. The coated aggregate is then placed in the block out in a maximum of 2 inch lifts.
8. The binder is heated to the appropriate pouring temperature and poured/flooded into the block out with the coated aggregate.
9. This process is repeated until the block out is filled.
10. A thin layer of anti-tracking material is placed upon the top surface to provide skid resistance.

Figure 8.26 Bridging Plate Installation

8.6.2.3 Strip Seals

A strip seal, also referred to as gland or membrane seal depending on the agency, is shown in Figure 8.27. Strip seals may need to be replaced if the headers are deteriorated or the extrusions are bent, loose, deformed or corroded. The seals may also need to be replaced if they are missing or significantly detached. To replace, first remove the remaining sections of the gland; heat may be necessary to melt the adhesives. Remove and replace deteriorated extrusions (see Figure 8.28 left) and re-anchor to new concrete header per steps provided in Section 8.6.1. Replace the gland using adhesives and tools suggested by the manufacturer, as shown in the example in Figure 8.28 right. A detailed strip seal replacement procedure published by Michigan DOT can be found at http://michigan.gov/documents/mdot/StructureMaintenanceBulletin4_Final_381367_7.pdf.

Figure 8.27 Strip/Gland/Membrane Seal

Figure 8.28 Removing the Old Gland (left) and Installation of New Gland (right)

It may be necessary to replace the entire joint system. A photo example and an example detail for installing a joint with a new strip seal are shown in Figure 8.29 and Figure 8.30. The complete header and steel extrusion system should be replaced as described in Section 8.6.1.

Figure 8.29 New Strip Seal Joint

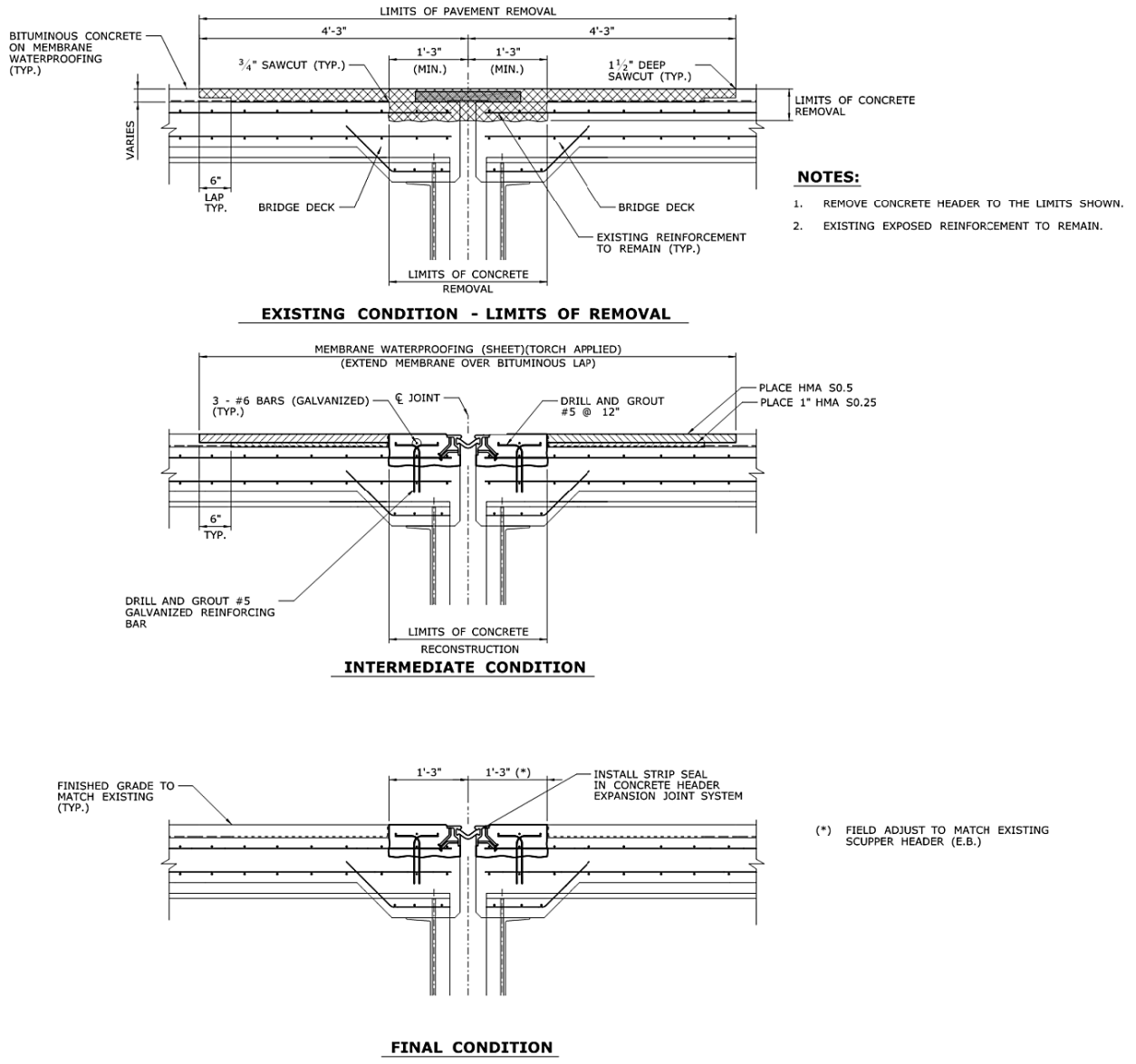


Figure 8.30 Detail for Installation of Joint with New Strip Seal

8.6.3 Rehabilitating Finger Joints

Maintenance of these joints can be required to restore the correct spacing between fingers, replace damaged or deformed fingers or restore deteriorated headers or anchorage systems.

Finger joints can be rehabilitated under stage construction since the plates can be cut and welded back together. The first step is to saw cut the concrete and remove the old anchors and create the block out for the joint system. The plates should be rehabilitated and new anchors fabricated and attached. Rebar should be threaded through the anchors prior to repositioning the plate into the deck. This plate should be repositioned back on the deck and the rebar previously threaded through the anchors connected to the existing deck rebar. It is important to install the drainage trough before both plates are positioned. The corresponding plate is then positioned and attached. Shims are used as spacers to provide the proper joint opening.

The joint spacing is determined by the span length and temperature range appropriate for the region, and the temperature at the time the plates are installed. The new plates and anchors should be recast taking care to ensure proper consolidation under the plates. A series of photographs are shown in Figure 8.31 through Figure 8.33 to illustrate the rehabilitation of a finger joint.

Figure 8.31 Damaged Finger Joint

Figure 8.32 Saw Cut and Remove Concrete to Create Block-Out

Figure 8.33 Rehab Plates and Add Anchors

Figure 8.34 Shims Used to Provide Proper Joint Spacing Prior to Casting the Joint

8.6.4 Replacing Joints with Alternative Joint Types

Although Compression Seals and Silicone Seals have been discussed in Sections 8.5.3 and 8.6.2, this section will discuss the process by which these joints can replace other types of joints, such as open joints or hot pourable seals (typically less-desirable small movement joints).

8.6.4.1 Upgrading Joint with Compression Seal

A water-tight compression seal can replace a small movement open joint. First step is to size the compression seal. The deck is then saw cut to the proper dimensions. Existing seals and other debris should be removed from the joint opening, as shown in Figure 8.35. Once the concrete has been sawcut and removed the joint edges should be sand blasted and cleaned. The seal should be coated with an adhesive which will serve as a lubricant until the material becomes tacky. Place the compression seal in the new opening, as shown Figure 8.36. This technique can be used to replace other joint types as long as the anticipated movement is in the same range as the compression seal (2-1/2 inches).

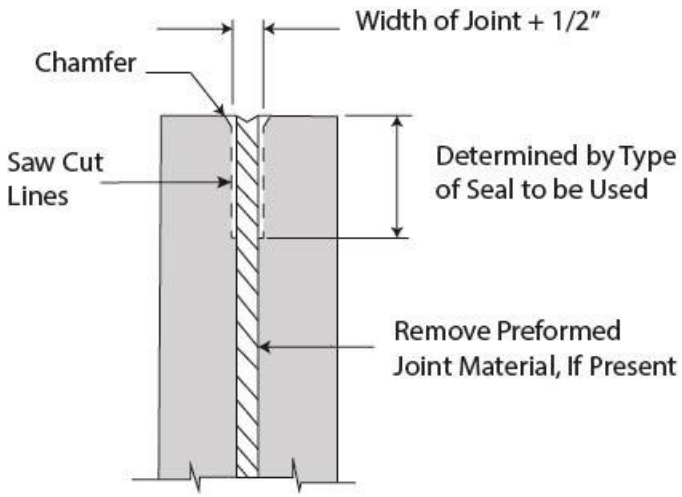


Figure 8.35 Joint Preparation for Compression Seal

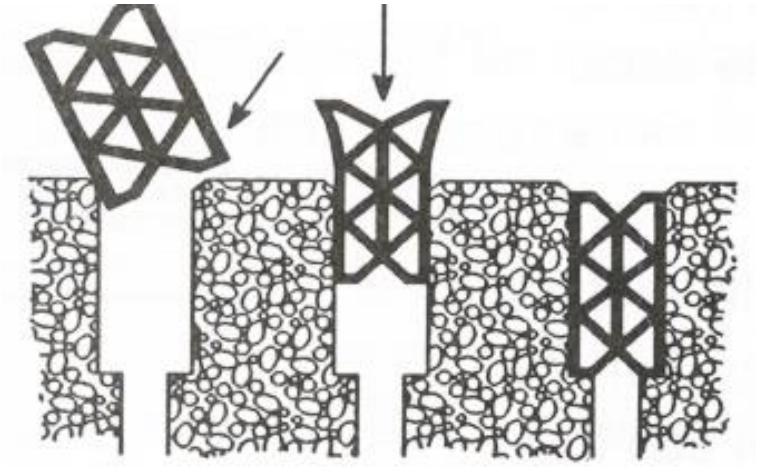


Figure 8.36 Insertion of Compression Seal

8.6.4.2 Upgrading Joint with Silicone Sealant

Silicone sealant can be used to replace hot poured sealants; this is desirable because the silicone system is more durable if applied properly. Other joint systems can be replaced with a silicone expansion joint system as long as the anticipated movement doesn't exceed the movement capacity of the silicone system (3 inches). A typical detail for the silicone expansion joint system was shown in Figure 8.3. This system can be used with or without asphalt overlays.

8.7 Elimination of Deck Expansion Joints

The best way to eliminate the problems associated with leaking bridge expansion joints is to eliminate the joint. In this section we will discuss various ways in which deck joints can be eliminated.

8.7.1 Slab over Backwall

Reconstructing the deck slab over the backwall can be a simple way to relocate the expansion joint to a more desirable location, particularly for small movement joints. Approximately 2 feet

of the slab and a portion of the backwall are removed. A schematic of this detail is shown in Figure 8.37. There are several ways to detail constructing a slab over a backwall; Figure 8.37 shows one way and is provided as an illustration. Care should be taken to preserve the deck and backwall rebar. The slab is then extended past the backwall. New rebar is provided for the deck extension. Often a portion or the entire approach slab is removed and reconstructed. In Figure 8.37, note there is no approach slab. There is typically a bond breaker provided atop the backwall to allow for the slab to move. Photographs of the construction of a slab over a backwall are shown in Figure 8.38 and Figure 8.39.

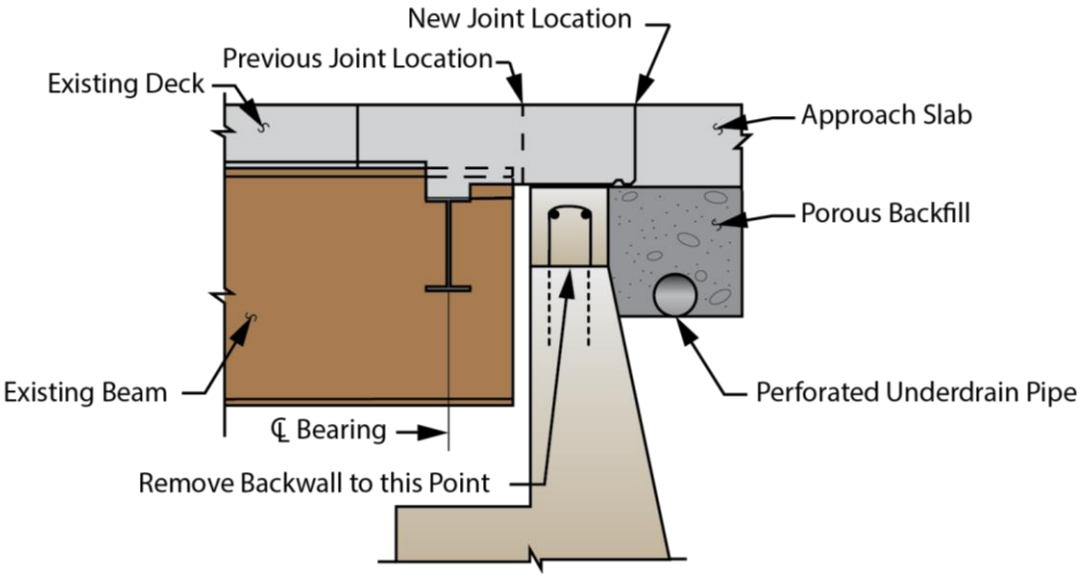


Figure 8.37 Slab Over Backwall Schematic

Figure 8.38 Construction Photo of Backwall Rebar

Figure 8.39 Construction Photo of Completed Slab over Backwall at Bridge Railing

When to Call the Engineer

Call the Engineer if slab over backwall is going to be done in conjunction with an approach slab.

8.7.2 Continuous Deck over Simple Span Piers (Link Slab)

Another relatively simple way to eliminate joints over a pier is by means of a link slab. A schematic detail is shown in Figure 8.40. A portion of the deck is removed from both spans sitting on the pier. The link slab portion of the deck is made non-composite by eliminating the shear studs; additionally the superstructure is ground smooth and a bond breaker is placed atop the girders. The length of slab to debond is L/20 (where L is the length of the span). The link slab can be designed in a simplified manner (Caner-Zia method) by treating the link slabs as a simple span (since the continuity provided by a continuous deck is negligible compared to the discontinuity of the stiffer superstructure). The length of this simple span is the length of portion of the slab that is debonded from the first span (L/20 span 1) + the length of the span that is debonded from the second span (L/20 span 2) + the distance between the girders. Moments in the link slab span can be computed by determining the end rotations in the adjacent spans which is a constant if they are multi-girder simple spans less than 100 feet. Moments for longer adjacent spans require computing end rotations for the simple spans under the calculated service loads. Reinforcement can be designed accordingly. Photographs of the construction of the link slab are shown in Figure 8.41 and Figure 8.42.

When to Call the Engineer

A structural engineer should be consulted to design the link slab reconstruction.

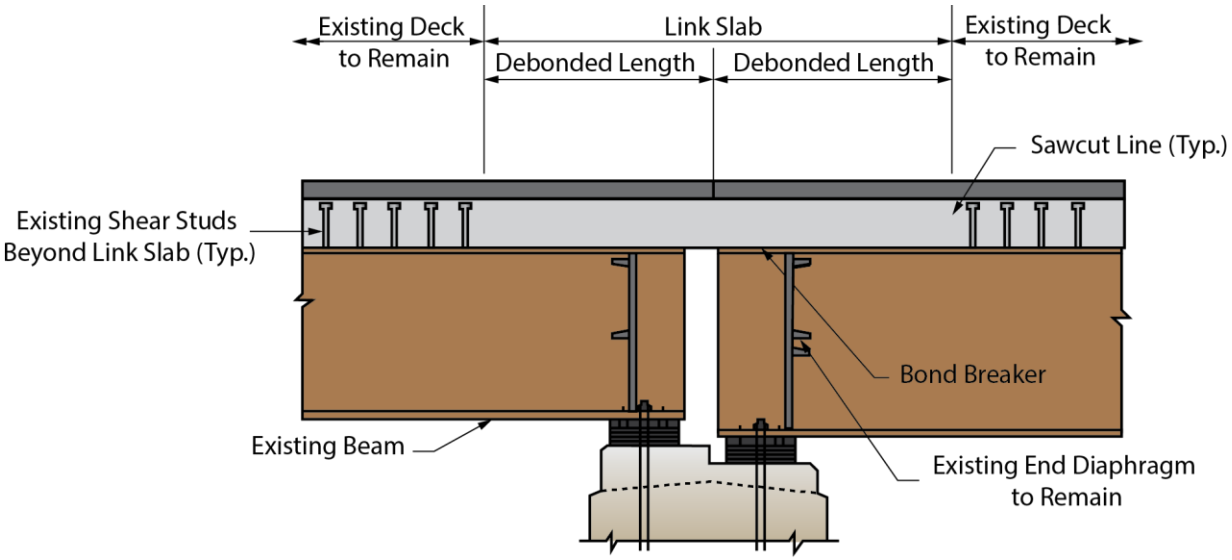


Figure 8.40 Link Slab, Or Continuous Deck over a Pier, Schematic

Figure 8.41 Link Slab Showing Bond Breaker and Elimination of Shear Studs

Figure 8.42 Link Slab Construction Showing New Slab Over the Pier and Link Slab Over the Girders

8.7.3 Integral/Semi-Integral Abutments

New bridges are now often designed using integral or semi-integral abutments. Although this is outside the Bridge Maintenance realm, it is important to be able to recognize these features and their purpose and function. The purpose of integral and semi-integral abutments is to eliminate bridge deck expansion joints, the movement associated with thermal expansion, and the material properties of the bridge components (namely creep and shrinkage) is accommodated in the abutments along with a joint in the pavement or adjacent to the approach slab.

8.7.3.1 Integral Abutments

These are bridges whose superstructure is cast integrally with the substructure. They typically have capped pile stub type abutments and a single row of vertical piles. A schematic is shown in Figure 8.43. As shown in the detail the end of the superstructure is encased in the abutment, which eliminates the expansion joints and bridge bearings. In integral abutments, the upper portion of the abutment which encases the beam ends is connected to the lower part of the abutment, which allows the abutment to rotate but not slide.

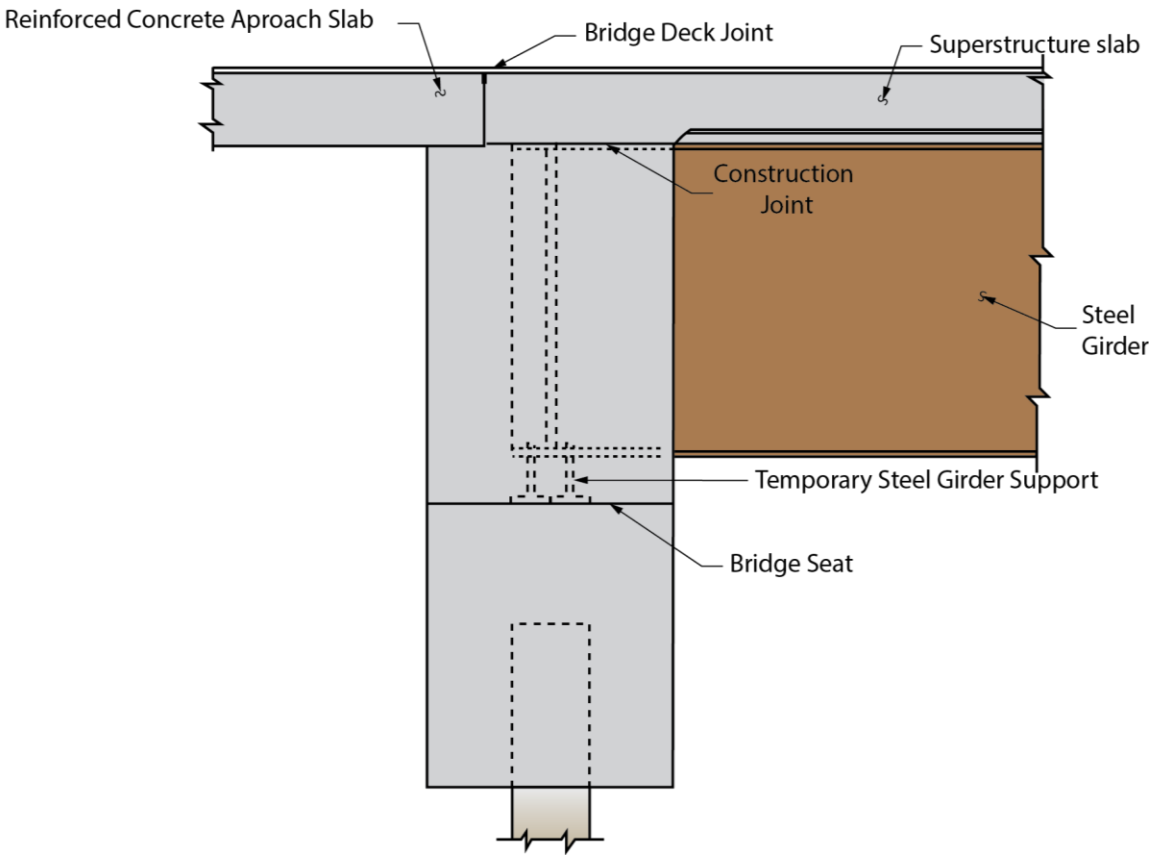


Figure 8.43 Integral Abutment Schematic

8.7.3.2 Semi-integral abutment

The semi-integral abutment is very similar to the integral abutment in that the superstructure is still cast into the substructure, thereby eliminating deck expansion joints, but the superstructure bears on the abutment thereby not eliminating the bearings. A schematic is shown in Figure 8.44.

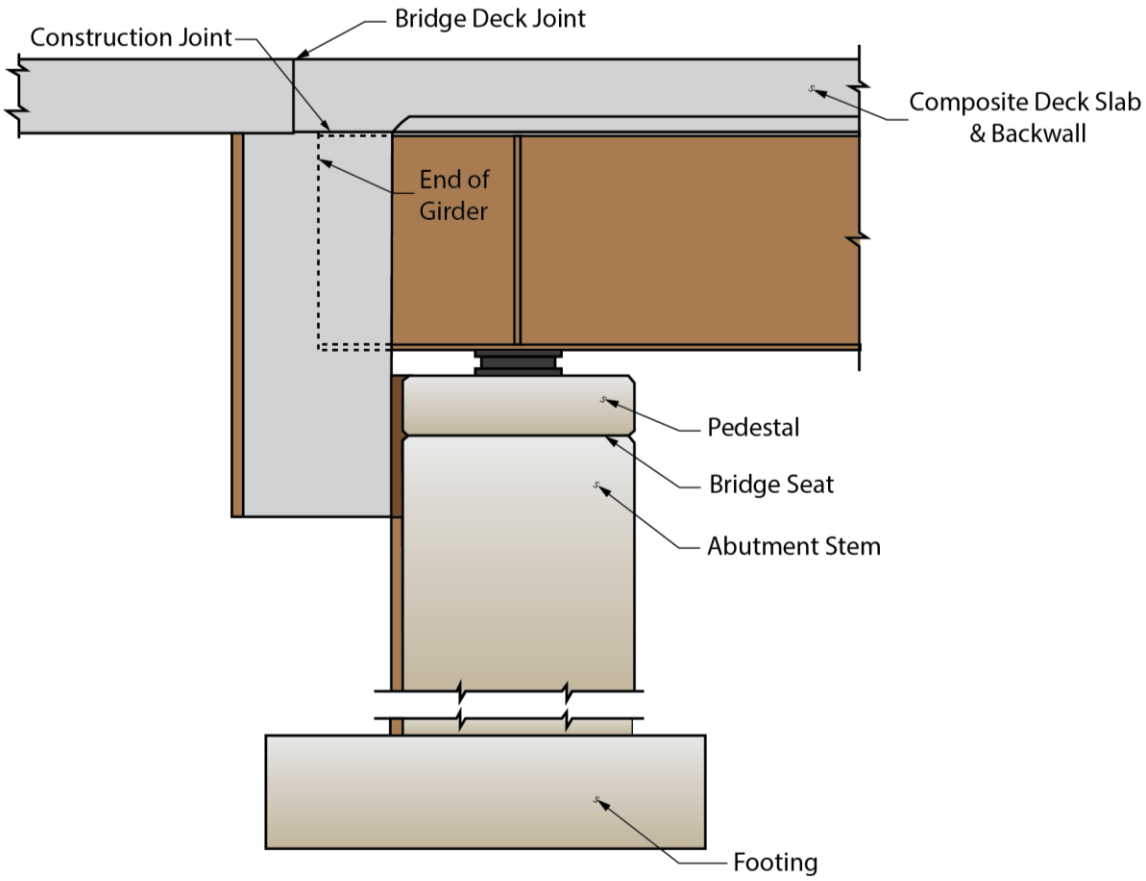


Figure 8.44 Semi-Integral Abutment Schematic

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Chapter 9 - Railing Systems and Safety Features

9.1 Purpose of Railing Systems

The fundamental purpose of a bridge railing system is to provide for public safety along the length and edges of a bridge. This includes redirecting errant vehicles back into the roadway without presenting a safety hazard when struck and ensuring the safety of bicyclists and pedestrians. Although railing systems generally do not provide structural strength to the bridge, the systems must be crashworthy. Therefore bridge railing differs from typical highway railing in that it is more rigid than flexible highway railing. Transition railing is used as the connection between the two as it provides a gradual change in the stiffness of the two railing systems.

Changes in the size and power of vehicles have spurred changes in railing design and requirements. The AASHTO Manual for Assessing Safety Hardware (MASH) is the latest practice for the crash testing of safety hardware devices for use on the National Highway System (NHS). It updates and replaces National Cooperative Highway Research Program (NCHRP) Report 350. Many states require that any new railing installed meet these standards. However much of the existing bridge railing in place today does not meet current code requirements because it was designed for the standards in place at the time of construction. It is generally recognized that replacement of all such railing would be a significant expenditure and therefore in many instances agencies allow it to remain until it requires replacement. Typically, if the damage incurred is significant, the railing system should be removed and replaced with a currently approved system. As applicable, repair may be an option when the damage is minor and the bridge still has a reasonable service life remaining.

To meet these intents, the following points are basic to good bridge railing systems:

- A roadside barrier on a bridge approach should provide a smooth transition to the bridge barrier. Exposed bridge rail ends or parapet walls should be avoided. After an emphasis was placed on smooth transitions by FHWA, the proportion of single-vehicle accidents at the end of bridge rails of parapets decreased. For example California and Texas went from 52 to 13 percent and 57 to 25 percent respectively.
- When a barrier or bridge railing installation is substandard; upon a significant project for repair, rehabilitation or reconstruction of the bridge and/or of its approaches; consideration should be given to either modifying the barrier to conform to an adequate system or replacing it with a current system meeting standards. Typical modifications would be to replace with approved crashed tested system such as a Jersey-type barrier (see Figure 9.1), if the structure can carry the additional dead load. Another alternative is to mount a metal guardrail across the structure (see Figure 9.2).
- If the railing has been severely damaged either by collision or weathering, complete replacement rather than repair may be more cost effective. Since most rails are from a standard design, it should be relatively easy to obtain plan details that might be needed for replacement.
- Collision-damaged railings should be repaired as soon as possible. Repair should include upgrading an old bridge railing that has been subjected to substantial damage to the current operational standard.

- In many cases, it is not possible to replace a collision damaged railing with one that meets current standards. In these cases, the damaged railing should be repaired/replaced in kind until the bridge is scheduled for rehabilitation or replacement.

Figure 9.1 Jersey Barrier and Guard Rail

Figure 9.2 Metal Beam Guard Rail

9.2 Components and Types of Railing Systems

There are many different shapes of railing used throughout the country. The selection of the railing is based on service level requirements such as design speed, average daily traffic volumes, how many trucks use the route, and site geometry. Other criteria such as functional and aesthetic considerations also play a role in the preference of railing to be used.

Railing systems at bridges and approaches are composed of four basic components:

- Bridge railings
- Transitions
- Approach guardrail
- Approach guardrail ends

These four basic components (shown in Figure 9.3 below) are designed to satisfy agency standards, which specify acceptable heights, materials, strengths, and geometric features.

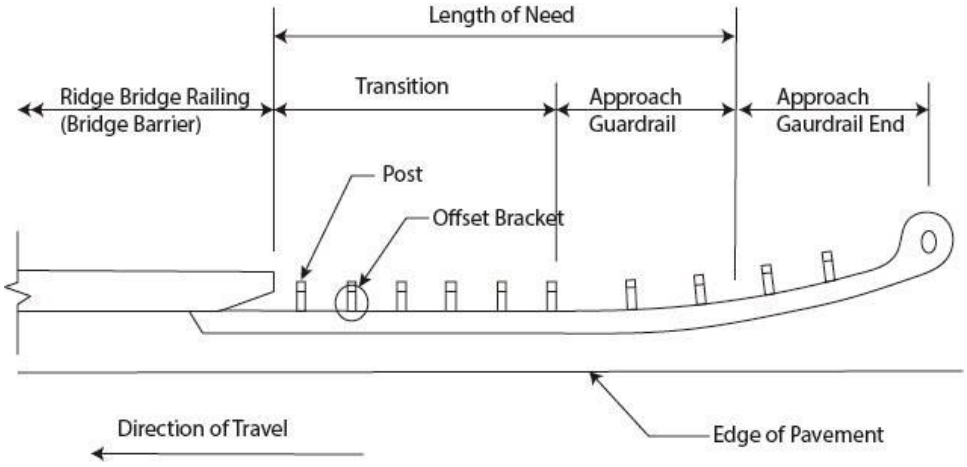

Figure 9.3 Schematic of Components of Railing Systems at Bridges

9.2.1 Bridge Railings

The function of bridge railing is to contain and smoothly redirect errant vehicles on the bridge. Many bridge rails could conceivably do this, but the safety of the driver and redirection of the vehicle must be taken into account.

9.2.2 Transitions

A transition occurs between the approach guardrail system and bridge railing (see Figure 9.3). Its purpose is to provide both a structurally secure connection to the rigid bridge railing and also a zone of gradual stiffening and strengthening of the more flexible approach guardrail system. Stiffening is essential to prevent “pocketing” or “snagging” of a colliding vehicle just before the rigid bridge railing end.

If, on impact, a redirective device undergoes relatively large lateral displacements within a relatively short longitudinal distance, pocketing is said to have occurred. Depending on the degree, pocketing can cause large and unacceptable vehicular decelerations. When a portion of the test vehicle, such as a wheel, engages a vertical element in the redirective device, such as a post, snagging is said to have occurred. The degree of snagging depends on the degree of engagement. Snagging may cause large and unacceptable vehicular decelerations.

9.2.3 Approach Guardrail

The approach guardrail system is intended to screen motorists from the hazardous feature beneath the bridge as they are approaching the bridge (see Figure 9.3). This approach guardrail screening is often extended in advance of the bridge so as to also keep motorists from any additional hazardous roadside features on the approach to the bridge.

Approach guardrail must have adequate length and structural qualities to safely contain and redirect an impacting vehicle within tolerable deceleration limits. Redirection should be smooth, without snagging, and should minimize any tendency for vehicle rollover or subsequent secondary collision with other vehicles.

9.2.4 Approach Guardrail Ends

The approach guardrail end is the special traffic friendly anchorage of the approach guardrail system. It is located at the end at which vehicles are approaching the bridge. Ground anchorage

is essential for adequate performance of the guardrail system. A special approach guardrail end is necessary in order to minimize its threat to motorists as another fixed object hazard within the roadside recovery area.

9.2.5 Common Types of Railing Systems

Some common railing shapes include:

- W-Beam Bridge Railing (Steel)
- Thrie Beam Bridge Railing (Steel)
- Metal Tube Bridge Rails (Steel, Aluminum)
- Vertical Concrete Parapet (Open or Closed) (Concrete)
- New Jersey Barrier (Concrete)
- F-Shape Barrier (Concrete)
- Single Slope (Concrete)
- Timber Bridge Railing (Wood)
- Laminate (Wood)

Photographic Examples of these common railing shapes are presented in Figure 9.4 thru Figure 9.12.

Figure 9.4 W-Beam Bridge Railing (Steel)

Figure 9.5 Thrie Beam Bridge Railing (Steel)

Figure 9.6 Metal Tube Bridge Rails (Steel, Aluminum)

Figure 9.7 Vertical Concrete Parapet (Open or Closed) (Concrete)

Figure 9.8 New Jersey Barrier (Concrete)

Figure 9.9 F-Shape Barrier

Figure 9.10 Single Slope Concrete Barrier

Figure 9.11 Timber Bridge Railing (Wood)

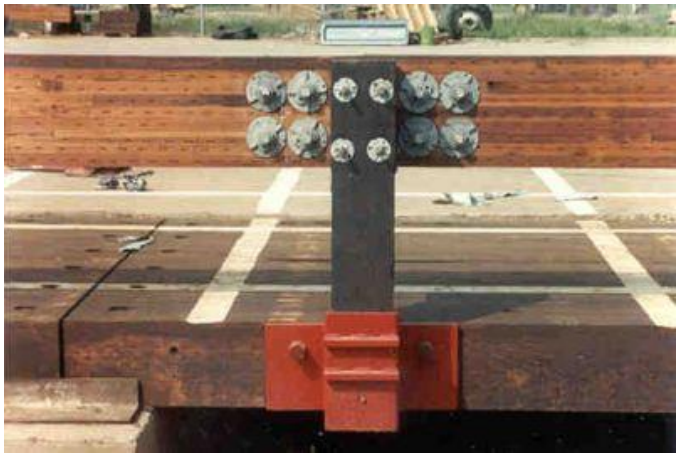

Figure 9.12 Laminate (Wood)

The purpose of this guide is to address the maintenance and possible repair of common materials of railing systems, as well as posts and anchors bolts as they are integral components of the system. A functional condition assessment of the entire damaged railing run is recommended prior to determining whether the entire railing system should be replaced in lieu of planning spot repairs.

When to Call the Engineer

Call the Engineer for a functional conditional assessment of the entire damaged rail run. The Engineer will determine whether the entire railing system should be replaced in lieu of planning spot repairs.

9.3 Preventive and Basic Maintenance of Railing Systems

Preventive maintenance can be thought of as the process of protecting the railing system from the unavoidable deterioration caused by moisture, de-icing chemicals, traffic vibration and the environment. Sometimes materials are flawed from the start, such as thin spots of galvanizing on steel or cracks in concrete due to improper curing when poured, which can cause early failure. It should be kept in mind that any railing system begins to deteriorate the day it is constructed. It is the purpose of preventative maintenance to slow the failure process as much as possible by using techniques and materials that are considered best practices. This section presents some methods to provide such maintenance; it is not meant to be all inclusive.

Prior to working on any bridge railing you should determine the extent of maintenance and supplies that will be required, as well as what signs and traffic controls are needed.

9.3.1 Concrete Maintenance

What To Look For

- Cracks
- Spalls
- Scaling
- Impact damage
- Deteriorating concrete/reinforcing
- Vegetation growing in cracks

Pressure washing of barriers to remove salt or other contaminants and dirt should occur on a regular basis. Many state DOT's recommend cleaning once every other year, early in the spring. Just like the deck of the bridge, the concrete railing is as susceptible to cracks and spalls from salt damage. Therefore it is essential to protect the surface and seal cracks with the appropriate sealant. The same methods of crack sealing can be used on the deck can be used on the railing. Application of concrete sealants and coatings will protect the reinforcing steel from corrosion by stopping or minimizing the intrusion of water and salt through the concrete. There are more options for protective coatings because the railing does not receive the same traffic wear as the deck. Some DOT's are using coatings, such as white epoxy, that not only waterproof the railing, but also enhance its visibility by reflecting light. These coatings should be applied before the railing becomes salt contaminated. They also must be reapplied on a regular basis, according to the manufacturer's recommendation. An example of coating a barrier is shown in Figure 9.13.

Figure 9.13 Application of Elastomeric Paint on Alkali-Silica Reactivity-Affected Barrier

Regular inspection for scaling, pop-outs and cracking should be performed. If scaling or pop-outs are found, the use of epoxies or methyl methacrylates should be used once the surface has been well cleaned. Small cracks (less than 1/16 in) should be cleaned and sealed with epoxies or high molecular weight methacrylate, whereas larger cracks should be routed out and sealed with flexible caulk. These defects should be sealed as soon as possible to ensure that salt does not have an opportunity to corrode the rebar.

If present on the structure, electrical conduits imbedded in concrete railings are exposed at the expansion joints. Slip joints in the conduit should be protected to prevent corrosion that may impair the function of the joint. Frequent inspection and maintenance of the protective coating is recommended.

In summary, preventative maintenance of concrete barriers includes:

1. Sealing the concrete (see Section 13.1.4).
2. Inspecting for and repairing scaling, pop-outs and cracks as soon as possible.
3. Reapplying the sealant as needed.
4. Keeping the railing free of salt, dirt, and debris by regular cleaning.

9.3.2 Steel Maintenance

What To Look For

- Impact damage
- Coating failure
- Corrosion
- Vegetation
- Loose hardware
- Rail at or near expansion joints

Most steel railing is galvanized and maintenance is minimal. Small scratches in the galvanized surface present little or no problem as the zinc will corrode first, keeping the underlying steel rust-free. However, where large portions of the zinc coating have been damaged, such as gouges from snow plow blades or over width loads, it is important to repair or touch-up the damaged section. Areas of spot rust should be sanded, cleaned and recoated to maintain the finish. A zinc-rich coating will be required. A photographic example of a galvanized steel railing is presented in Figure 9.14.

Figure 9.14 Galvanized Steel Rail

Other areas of concern are where the steel is in contact with concrete or other metals. To prevent corrosion these areas should have an insulating material between the two in order to protect the steel.

A regular check of the fasteners is recommended. Any loose connections should be tightened. Finally, a regular cleaning, such as pressure washing, is recommended to keep connections free of debris.

In summary, preventative maintenance of steel railing includes:

1. Touching up any minor damage with zinc-rich paint.
2. Inspecting and maintaining fastener tightness.
3. Ensuring a bond breaker is between the steel and any other material it contacts.
4. Keeping the railing free of salt, dirt, and debris by regular cleaning.

9.3.3 Aluminum Maintenance

Aluminum weathers to a dull grey appearance and is highly resistant to atmospheric corrosion. This finish provides good corrosion resistance and an almost unlimited life expectancy. However, pitting corrosion can occur on aluminum surfaces frequently in contact with a humid environment. Generally, this is only an aesthetic consequence. Accumulation of dirt and debris on surfaces can cause a reduced durability due to the extended exposure to moisture. Dirt and debris should be removed on a regular basis. An example of an aluminum railing is presented in Figure 9.15.

Figure 9.15 Aluminum Railing

Oxidation protection is required at the contact surface with other materials and this should be checked on a regular basis. Steel to aluminum contact surfaces should be caulked with an elastic, non-staining blend of water-repellant oil, asbestos fiber, and flakes of aluminum (metal or other suitable materials) to prevent corrosion.

A regular check of the fasteners is recommended. Any loose connections should be tightened.

The contact surface of each aluminum railing post attached to concrete should be separated from the concrete with a nonreactive bedding material such as 30 pound nonperforated, asphalt-saturated felt; a galvanized or painted steel plate; or an elastomeric caulking compound. If a grout pad is used under the base plate, any damage to the pad should be repaired. If grout comes in contact with the surface of the aluminum, it will cause visible spots. These are difficult to remove and for this reason aluminum should be protected onsite.

In summary, preventive maintenance of aluminum railing includes:

1. Inspecting and maintaining fastener tightness.
2. Ensuring a caulk layer is between the aluminum and any other material it contacts.
3. Keeping the railing free of salt, dirt, and debris by regular cleaning.

9.3.4 Wood Maintenance

Wood is a durable bridge material, but over extended periods it may be subject to deterioration from decay, insect attack, or mechanical damage. Protective systems are a necessity for wood railing systems to resist decay. Most wood will have been pressure treated prior to construction and therefore require minimal maintenance.

Moisture is a prime enemy of wood. Periodic washing will remove dirt and debris that hold moisture. Mildew shows up as green or black spots on bare wood or paint. It can be removed with a solution of sodium hypochlorite (bleach) and water. Identify areas with visible wetting or high moisture and take corrective action to eliminate the source. Checks, splits and damaged areas should be treated as soon as possible. Field drilled holes should be treated with preservatives before installing bolts and other hardware.

For painted rail, a routine maintenance schedule should be followed. In order for the paint to serve its purpose, the surface of the wood has to be properly prepared. The existing railing should be pressure washed to remove flaking or chipped paint and any residue. It may be

necessary to spot sand at some locations. A sealant primer should be applied which is compatible with both the wood preservative and the paint. This will reduce the risk of further bleeding and need for repainting. Paints used for wood railing protection are usually oil-based, oil-alkyd or latex-based. Oil-based paints provide the best protection from moisture but are not very durable. Oil-alkyd paints have more durability than oil-based paints but contain lead pigments which may cause various health hazards. Latex-based paints are very flexible and resistant to chemicals.

Corrosion damage can occur if the wood is in contact with uncoated fasteners containing iron. All fasteners should be galvanized. Check and repair any damage to galvanized parts.

The wood bridge post is very vulnerable to ultraviolet light degradation. The exposed end grain of the post also allows for frequent wetting and drying cycles which hasten deterioration. Consider using a post cap which shields the wood post from UV light while sheltering the end grain from wetting at the same time.

In summary, preventive maintenance of wood railing includes:

1. Maintaining a coating of paint and sealant, if the railing is painted.
2. Inspecting and maintaining fastener tightness.
3. Ensuring all fasteners in contact with wood surfaces are galvanized.
4. Keeping the railing free of moisture, dirt, and debris by regular cleaning.

9.4 Repair and Rehabilitation of Railing Systems

When to Call the Engineer

If the structural elements of the bridge to which the rail is attached are damaged, such as the bridge deck, wingwalls, back walls, etc., the Engineer should be consulted.

Most states require that bridge railing with damage or deterioration that may prevent containment and/or redirection of errant vehicles traveling at the posted speed limit be replaced. Any replacement railing generally will be required to be upgraded to meet current standards. Often this means that the entire run of railing will be replaced.

The FHWA recommends giving consideration to the replacement of substandard bridge rails as part of any bridge rehabilitation, reconstruction, or replacement project. The addition of a continuous section of standard guardrail in front of and attached to the existing bridge railing is a very common manner of upgrading substandard bridge rail.

However there may be limited instances where repair is acceptable. The remainder of this chapter will address some general repairs which may be taken prior to replacement.

Some critical actions that should be taken before the actual repair include:

- Upon notification of damage that leaves the railing nonfunctional, and if it cannot be repaired immediately, warn traffic of the hazard by putting out temporary warning

devices in accordance with the Manual on Uniform Traffic Control Devices (MUTCD), such as drums, vertical panels, cones, or other devices.

- Contact your local Dig Safe utility protection well in advance of going to the site.
- Take enough signs and channelizing devices to the site to properly mark the repair zone. When your agency uses an arrow board and/or a shadow truck or Truck Mounted Attenuator, be certain there is enough equipment and personnel to handle these items.
- Set up the Temporary Traffic Control Plan for the work area to repair the railing in accordance with the MUTCD or your agency requirements.
- Ensure all workers wear equipment as provided by the Occupational and Safety Administration (OSHA) or local requirements, such as safety visibility vests, safety glasses, and protective-type shoes.
- Ensure all equipment and procedures conform to OSHA guidance.

9.4.1 Concrete Repair

If there is minor delamination of the surface, typically caused by the corrosion of reinforcing steel, repairs can be made. In general, the process is to remove the unsound concrete; sawcut around the perimeter; and remove and patch with fast setting patch material. If there are thin areas (1-in or less), they can be patched with trowelable mortar.

For widespread surface deterioration, typically caused by corrosion of the reinforcing steel and/or poor quality concrete, the repair is more substantial. It may be possible to sawcut around the failed areas, perform a full-depth removal of the unsound areas, then recast in-kind. If the deterioration is significant it may be more cost effective to replace entire railing.

If one or more sections of railing are broken, the procedure for concrete railing repair includes the following steps:

Suggested Procedure

Concrete Rail Repair

1. Assuming that the rail will be replaced "in kind," which is a decision that should be made by the engineer; the plans should be obtained for use in constructing the new sections.
2. Mark off areas of rail to be replaced.
3. Sawcut to a depth of at least ¾ - 1 inch. Be careful not to cut through reinforcing steel.
4. Remove concrete in sections of rail to be replaced with a chipping hammer. Do not place chipping hammer on reinforcement.
5. Use sand blasting equipment to remove any rust from exposed reinforcing steel and to clean the exposed concrete surfaces. Touch up epoxy bars.
6. Replace any missing or corroded steel with new bars providing proper overlap lengths. Consult an engineer as needed. Overlap requirements may require a larger area to be chipped out, or drill and epoxy anchor bars, or mechanically splice.
7. Form new sections to conform to rail dimensions on plans.
8. Apply form release agent to forms.
9. Place concrete.
10. Cure concrete for at least 72 hours.
11. Finish surface with a rubbing stone to match existing rail and clean up the job site.

When to Call the Engineer

Call the Engineer when deciding if a rail will be replaced “in kind”.

9.4.2 Steel Repair

Metal railing is fabricated in standard sections and most bridges are designed with one of the standard sections. It is a good idea to have an inventory of the common sections used in a state, with necessary hardware, for replacement needs. Repair and replacement of steel pipe and tubular railings should be made as follows:

Suggested Procedure

Repair and Replacement of Steel Pipe and Tubular Railings

1. Collision-damaged steel railings generally have to be replaced. Occasionally this type of railing can be straightened and repaired. When delays in receiving new or replacement parts are encountered, a temporary railing repair should be made to protect the public.
2. Loose anchor bolts and connections should be tightened. If corrosion is present, painting or galvanizing procedures should be followed.
3. Rust stains around the perimeter of steel rail posts or anchor bolts imbedded in concrete indicate corrosion. Corroded areas should be thoroughly cleaned and painted. Railing components that have section loss should be repaired or replaced.
4. Hot-dip galvanizing is recommended for new, replacement, or existing railings. Zinc-rich paint may also be used. Painted railing will require frequent repainting when located in industrial or marine or salt laden environments. Care should be taken to touch up the protective coating after tightening the nuts on anchor bolts and other rail connections.
5. Damaged anchor bolts should be repaired or replaced as required. Refer Section 9.4.5 Anchorage Repair.

9.4.3 Aluminum Repair

Aluminum railing damaged by collision should be replaced promptly. New railing sections are generally preferred because of the high degree of experience required in straightening or welding aluminum. Consideration should be given to replacement with "New Jersey" type concrete railing. Steps similar to steel repair can be taken. Do not attempt to straighten, hot dip galvanize or use zinc-rich paint.

9.4.4 Wood Repair

Any deterioration of railing and post members will reduce effectiveness in the event of an accident. In some cases it may be acceptable to splice the members with galvanized steel plates or install protective jackets to restore structural unity and prevent further splitting. Generally if wood railing or posts have been damaged by collision or exhibit section loss from deterioration, they will need replacement. Steps similar to steel repair can be taken. Do not attempt to straighten, hot dip galvanize or use zinc-rich paint.

Wood approach guardrail does not meet minimum criteria for strength, continuity, or performance for bridges on the National Highway System. However, it may be found on some state or local bridge approaches. They are frequently hit by errant vehicles and must be replaced. For the posts, typical embedment is between three to three and a half feet. Posts can be removed using a chain attached to a tractor bucket or dump truck body. If the wood posts are entirely broken by severe impacts, it will be necessary to dig around the remaining stub to a depth deep enough to wrap a chain around the lower section of the post for removal.

There are several ways that the new posts can be reset. Wood posts generally are not driven into the ground; however in some cases it may be possible to drive a new wood post in the existing hole. Otherwise a hole is either drilled or excavated and the new post set plumb. Backfill the hole with compactable materials in 6 inch lifts and compact with pneumatic equipment to optimum compaction. After posts are installed to correct positions, blocks and guardrails may be installed.

9.4.5 Anchorage Repair

When to Call the Engineer

Call the Engineer if anchor bolts are damaged to determine if they can be repaired or replaced. Call the Engineer if the concrete in which the anchor bolts are embedded has been damaged.

In some instances, anchor bolts may require extension to accommodate the installation of a new railing. In other cases, the anchor bolts are damaged or there is damage or deterioration of the deck affecting the integrity of the anchorage. There are three common types of replacement anchors including adhesively bonded (Figure 9.16 left), through bolted (Figure 9.16 right), or a combination of the two. These should be considered temporary repairs until the bridge deck is adequately repaired or replaced. Note that the FHWA discourages the use of adhesive anchors in tension applications: http://www.fhwa.dot.gov/bridge/t514030.cfm. Additionally, adhesive anchors are not approved by some DOT's.

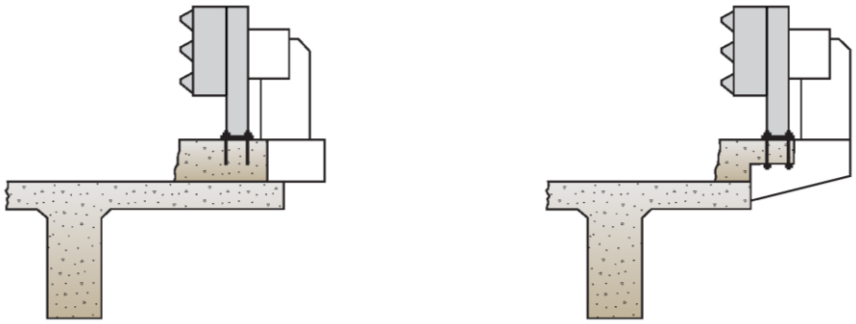

Figure 9.16 Anchor Examples. Bonded Anchor (left). Through Anchor (right).

Where anchor bolts must be extended, chip away the concrete around the bolt to allow welding below the finished surface of the concrete. Old bolts should be cut and ground at an angle to provide for a stronger weld. Replace damaged concrete or concrete that has been removed as repairs are made. Large areas justify concrete replacement after the railing has been repaired.

Post-installed anchors are those that are installed into hardened concrete. There are two main types typically used for railing installations or retrofits. These are through bolted anchors and bonded anchors. The bonded anchor systems typically include three different bonding options (discussed later).

Through bolted anchors are simply anchors that extend entirely through the thickness of the deck. Typical installation includes carefully field coring holes through the superstructure to nominally accommodate the bolts. A backer plate is placed on the bottom of the deck and a base plate is positioned on the top. Threaded anchor rods are inserted and tightened. Beveled washers may be required to allow appropriate tightening.

Bonded anchors differ in that the anchor rod is able to withstand loading due to the adhesion or bond created between the deck and the bonding agent. Consequently, adequate performance of the anchors can only be achieved when anchors are installed according to the manufacturer's published installation procedures. Once installed, it is recommended that at least one of them be tested for pullout resistance. Bonded anchor systems include the following:

1. Drill and Bond Dowel: Magnesium phosphate (mag-phos) is used as a bonding agent. Within three hours, mag-phos concrete hardens or cures. It does not require any special treatment during curing. It also takes about three days to develop its full strength. Mag-phos also has a relatively high resistance to pull out and has minimal shrinkage, therefore less cracking. However it cannot be used for galvanized anchors or come into contact with zinc or aluminum. It will be less effective in cracked concrete.
2. Drill and Grout Dowel: Neat Portland cement paste (grout) is used as a bonding agent. Generally, cement grout is less expensive than mag-phos concrete. However, the grout has to be cured for at least three days during which time the dowels should not be disturbed. The grout normally develops half of its strength in three days, and develops full strength in about 28 days. In addition, grout has a tendency to shrink, leading to cracks.
3. Drill and Bond Dowel (chemical adhesive): A chemical adhesive or a cartridge epoxy is used as a bonding agent. These adhesives are thicker than the other two bonding agents. This helps the adhesive stay better in a drilled hole. It has a relatively quick setting time which may differ slightly between manufacturers. It has little shrinkage resulting in less cracking. However, it requires stringent quality control and quality assurance testing, particularly since it may slowly deformation over time.

Installation instructions particular to the type of anchor will specify the drilling method, hole cleaning procedure, how to install the bonding agent and the metal anchor in the drilled hole, and the care to be taken until the bonding agent has cured. Under no circumstances should these types of bonded anchors be used through hollow sidewalk cores. In general, drill and grout and drill and bond (mag-phos) should be used only in holes drilled between 90 degrees and no less than 20 degrees (or 3:1) to the horizontal. In horizontal holes, chemical adhesives should always be used since the adhesives are less likely to run out of the holes. For example, these may be used in deck overhangs where horizontal holes may be the only option due to a thin or narrow deck. This anchorage system is not recommended for use in overhead applications.

For the above anchor treatments, be sure to repair any damage in the deck due to improperly cored holes. Only install guardrail retrofits on structures that have the capacity to support them, as well as withstand potential impact. For instance, do not retrofit or replace new anchors on a severely deteriorated deck. Confirm clearances before field coring into the existing deck.

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Chapter 10 - Deck Drainage

10.1 Common Types of Deck Drainage Systems

The bridge deck drainage system includes the bridge deck, sidewalks, railings, gutters, scuppers/inlets and open or closed piping. The primary objective of the drainage system is to remove runoff from the bridge deck before it collects in the gutter, shoulder or lane thus exceeding the allowable design spread as defined by the bridge owner. Properly designed bridge deck drainage should incorporate features to facilitate and minimize maintenance activities. The benefits of a properly designed and maintained deck drainage system include the following:

- Efficient removal of water from the bridge deck which decreases the risk of hydroplaning and enhances public safety
- Long-term maintenance and service life of the bridge is improved
- Preservation of structural integrity of the bridge
- Enhanced aesthetics by elimination of superstructure and substructure staining
- Minimized or eliminated erosion on bridge end slopes

10.1.1 Open vs. Closed Deck Drainage Systems

Open deck drainage systems are typically horizontal or vertical penetrations thru the bridge barrier/curb or the bridge deck at the bridge barrier/curb line. Horizontal penetrations (drainage slots) are typically formed as part of the bridge barrier curb construction and closely spaced based on the small opening of the slot. This system is prone to clogging with debris and often requires constant maintenance. In addition, this system allows chloride and chemical-contaminated runoff to come in direct contact with the bridge deck fascia overhang and superstructure below. Such runoff may result in premature spalling of concrete or corrosion of steel. Vertical penetrations are typically constructed with a circular hole thru the bridge deck, or a scupper or inlet with pipe extension thru the bridge deck. Open deck drainage systems are typically only used where there are no concerns with environmental impacts or runoff below the bridge and possible flooding, and no active facilities below the bridge. Figure 10.1 shows the two open deck drainage systems described above.

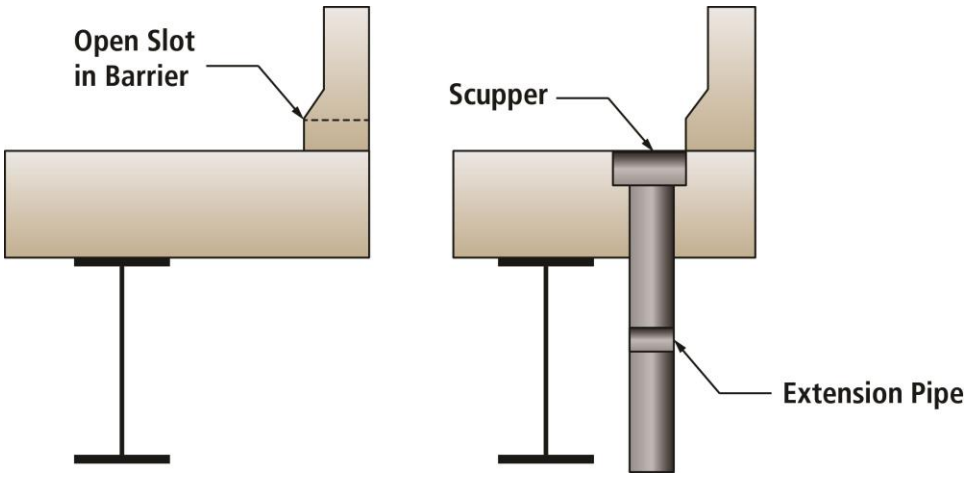

Figure 10.1 Schematic of an Open Deck Drainage System. Horizontal Penetration (left) and Vertical Penetration (right).

Closed deck drainage systems typically are comprised of scuppers or inlets at the deck surface with a closed piping system which extends from the base of the scupper/inlet down a substructure element (Pier or Abutment) and into a ground based drainage inlet, as shown in the schematic in Figure 10.2. The pipe is typically attached using supports to the substructure element and includes cleanouts at proper locations for maintenance. Closed drainage systems are typically used where there are concerns with environmental impacts, such as flooding below the structure and possible erosion, or active facilities below the structure. A photograph of a closed deck drainage system is shown in Figure 10.3.

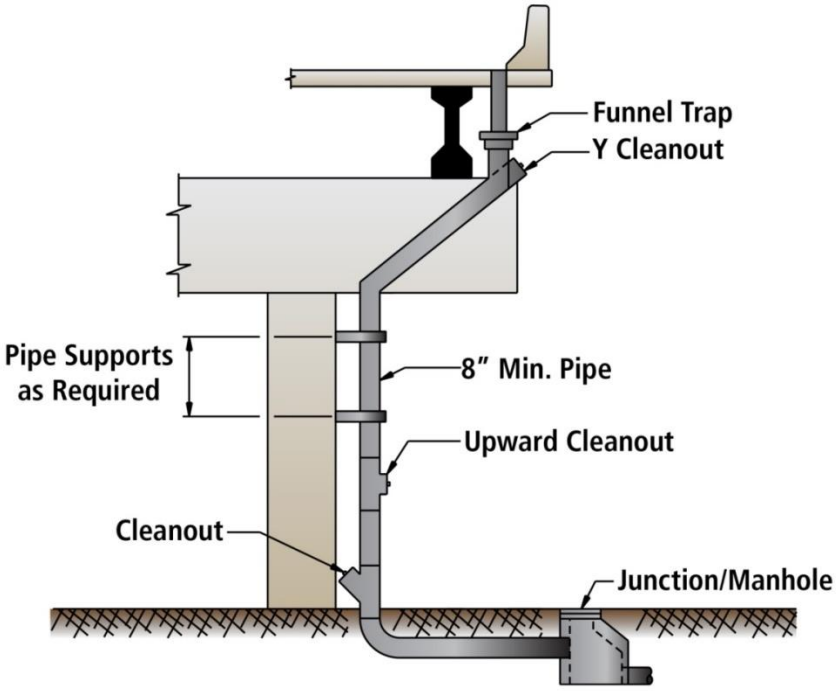

Figure 10.2 Schematic of a Closed Deck Drainage System

Figure 10.3 Photo of a Closed Bridge Drainage System – Fiberglass (Courtesy of FRPbridgedrainpipe.com)

10.1.2 Scuppers

A scupper is defined typically as a vertical hole through a bridge deck for the purpose of deck drainage. However, a horizontal opening in the curb or barrier is also considered a scupper as it serves the same purpose of deck drainage. A scupper can be as simple as a circular penetration in the bridge deck, with or without an insert pipe in the bridge deck. Or it can be a combination of a small steel-grated casting typically with a steel pipe extended below the superstructure. Examples of vertical and offset scuppers are presented in Figure 10.4 and Figure 10.5, respectively. Additional example scupper details are provided in the drawings in Figure 10.6 and Figure 10.7.

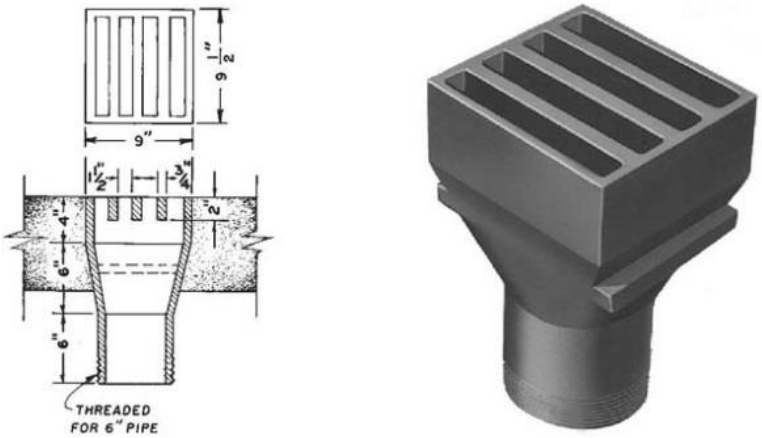

Figure 10.4 Heavy Duty Vertical Scupper with Integral Cast Bars (Courtesy of Neenah Foundry Company)

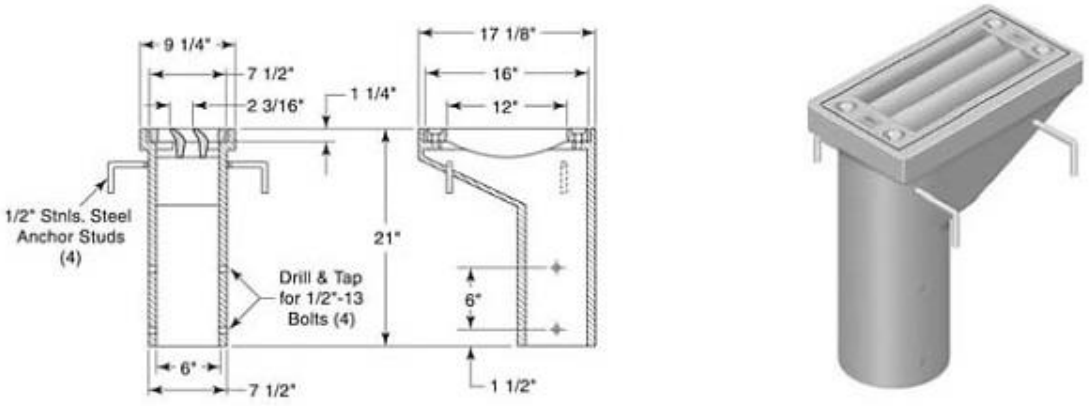

Figure 10.5 Offset Scupper with Frame and Bolted Grate
(Courtesy of Neenah Foundry Company)

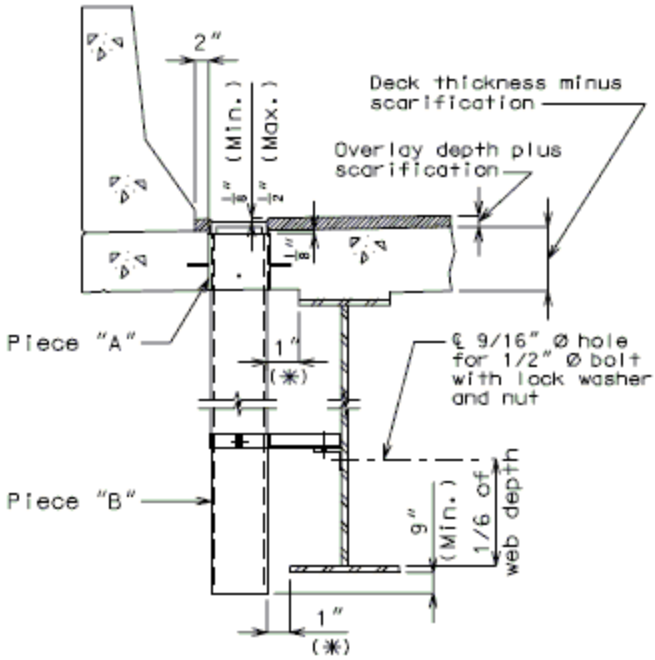

Figure 10.6 Example Bridge Scupper Detail with Pipe Extension
(Courtesy of the Missouri DOT)

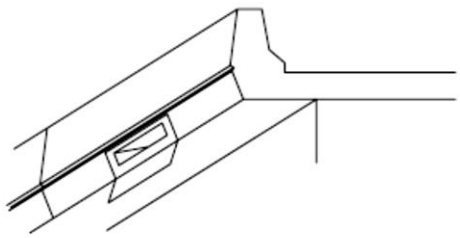

Figure 10.7 Example of a Horizontal Bridge Scupper through a Bridge Parapet
(Courtesy of the Maryland DOT)

10.1.3 Inlets

An inlet may be defined as a bridge inlet or bridge end inlet. Some agencies use the term bridge inlet and scupper interchangeably. In general, a bridge inlet has a larger opening than a scupper and is comprised of a casting or fabricated inlet base with a cast or fabricated grate. Example drawings of a bridge inlet are presented in Figure 10.8 and Figure 10.9. The larger bridge inlets allow for collection of greater runoff and typically collect less debris and require less maintenance. A bridge end inlet is usually a precast roadway inlet with grate which is placed at the end of the bridge approach slab to collect bridge runoff on bridges which have no bridge deck scuppers or inlets, or for longer span bridges where spacing between inlets may be long. The drawings provided in Figure 10.10 and Figure 10.11 provide examples of bridge end inlet design.

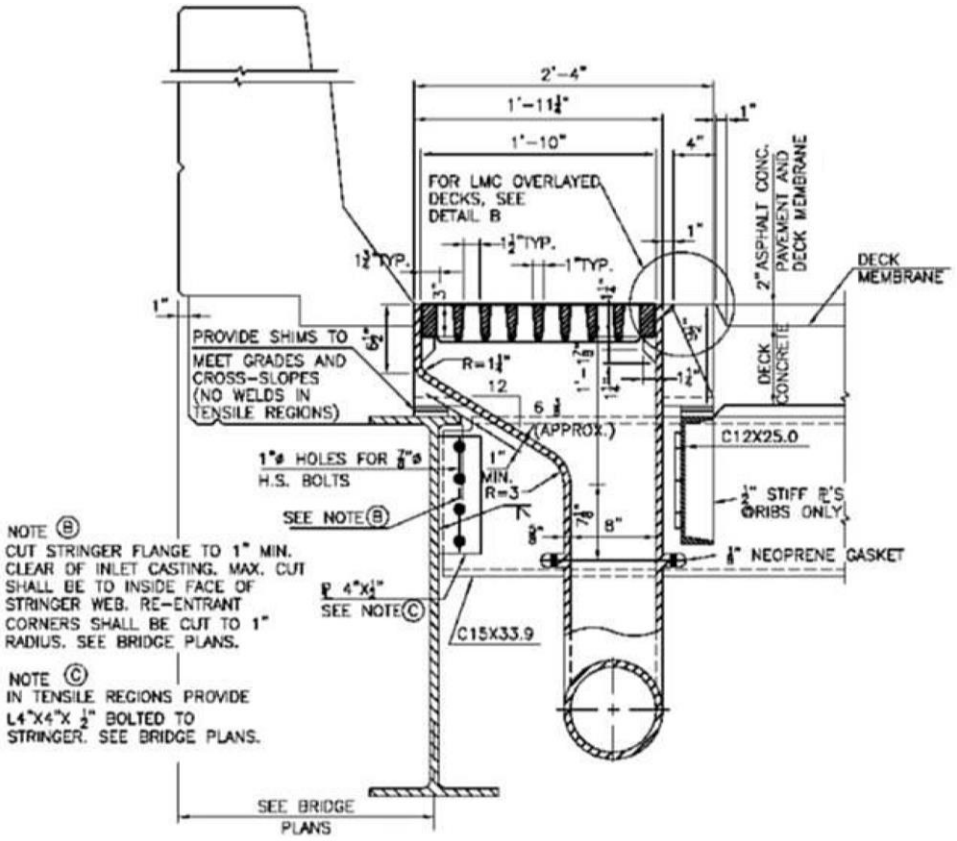
2'-4"
1'-11½"
1'-10"
1"
4"
FOR LMC OVERLAYED DECKS, SEE DETAIL B
1½" TYP.
1½" TYP.
1" TYP.
1"
2" ASPHALT CONC. PAVEMENT AND DECK MEMBRANE
DECK MEMBRANE
DECK CONCRETE
PROVIDE SHIMS TO MEET GRADES AND CROSS-SLOPES (NO WELDS IN TENSILE REGIONS)
R=1½"
12
6 ¾" (APPROX.)
C12X25.0
1"ø HOLES FOR ⅞"ø H.S. BOLTS
1" MIN. R=3
SEE NOTE Ⓑ
½" STIFF PL'S @ RIBS ONLY
8"
⅜" NEOPRENE GASKET
NOTE Ⓑ
CUT STRINGER FLANGE TO 1" MIN. CLEAR OF INLET CASTING. MAX. CUT SHALL BE TO INSIDE FACE OF STRINGER WEB. RE-ENTRANT CORNERS SHALL BE CUT TO 1" RADIUS. SEE BRIDGE PLANS.
℄ 4"x½" SEE NOTE Ⓒ
C15X33.9
NOTE Ⓒ
IN TENSILE REGIONS PROVIDE L4"X4"X ½" BOLTED TO STRINGER. SEE BRIDGE PLANS.
SEE BRIDGE PLANS

Figure 10.8 Example Bridge Inlet Detail – Cross Section View (Courtesy of the New Jersey Turnpike Authority)

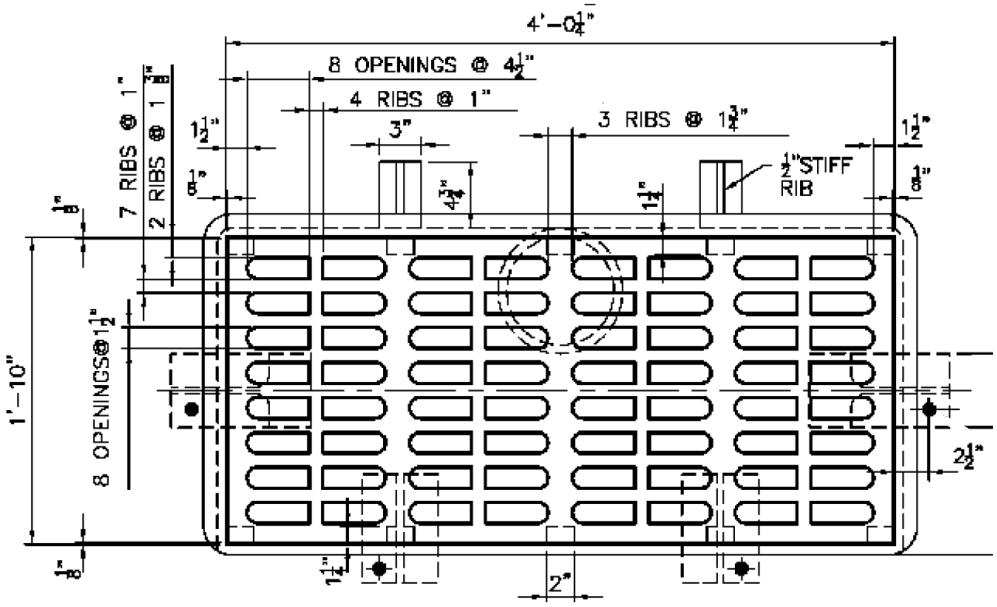

Figure 10.9 Example Bridge Inlet Detail – Plan View (Courtesy of the New Jersey Turnpike Authority)

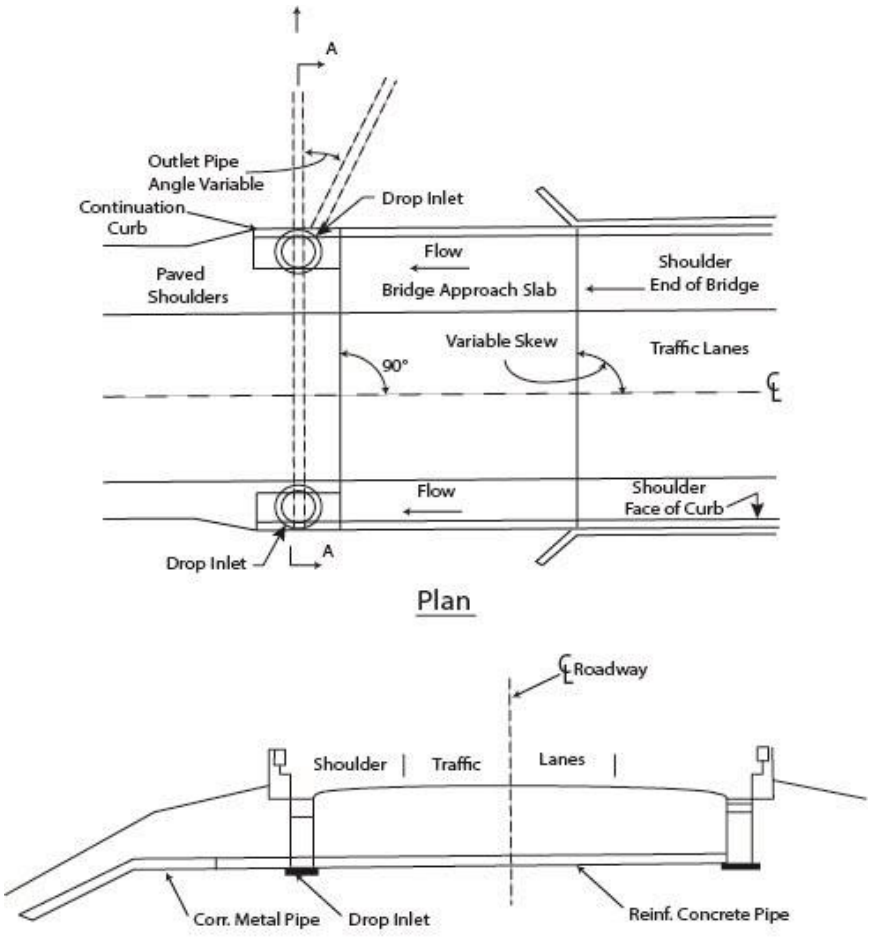

Figure 10.10 Bridge End Inlet Plan & Section Example Drawing

When to Call the Engineer

Call the Engineer if there are sink holes, voids, or other evidence of erosion in the approach embankment in the area of drop inlets or buried pipe locations.

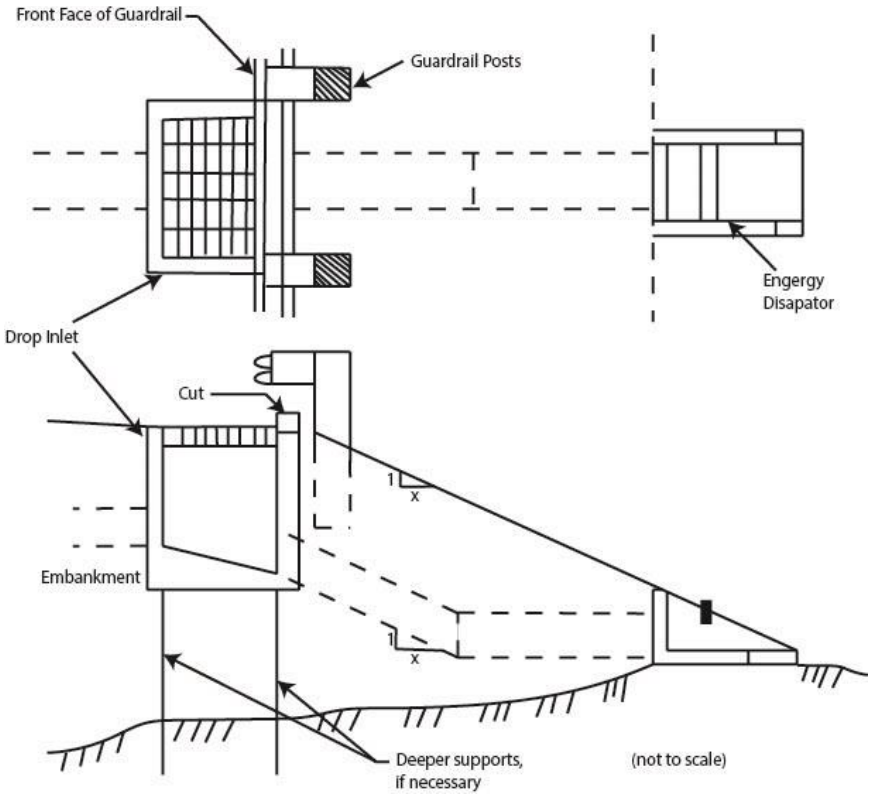

Figure 10.11 Bridge End Inlet Detailed Plan & Section Example Drawing

10.1.4 Troughs

A drainage trough is typically used under open deck joints, such as finger joints, to divert runoff and associated debris away from underlying superstructure, bearings and substructure members. Drainage troughs on older bridges were made of steel and required significant maintenance based on flat slopes which allowed for the easy collection of debris and replacement or repair due to corrosion. A photographic example of a steel drainage trough is shown in Figure 10.12. Drainage troughs on new bridges are typically made of elastomeric sheet, as shown in the schematic in Figure 10.13. These troughs are considered self-cleaning if designed and detailed properly. Examples are shown in Figure 10.14 and Figure 10.15, as they are flexible and will move laterally and vertically under bridge expansion/contraction and live load. Photographic examples of a clean elastomeric sheet trough and also a clogged trough are shown in Figure 10.16 and Figure 10.17, respectively.

Figure 10.12 Unmaintainable Steel Trough and Drainage Deflection Plates (Courtesy of the New Jersey Turnpike Authority)

Figure 10.13 Schematic of a Drainage Trough

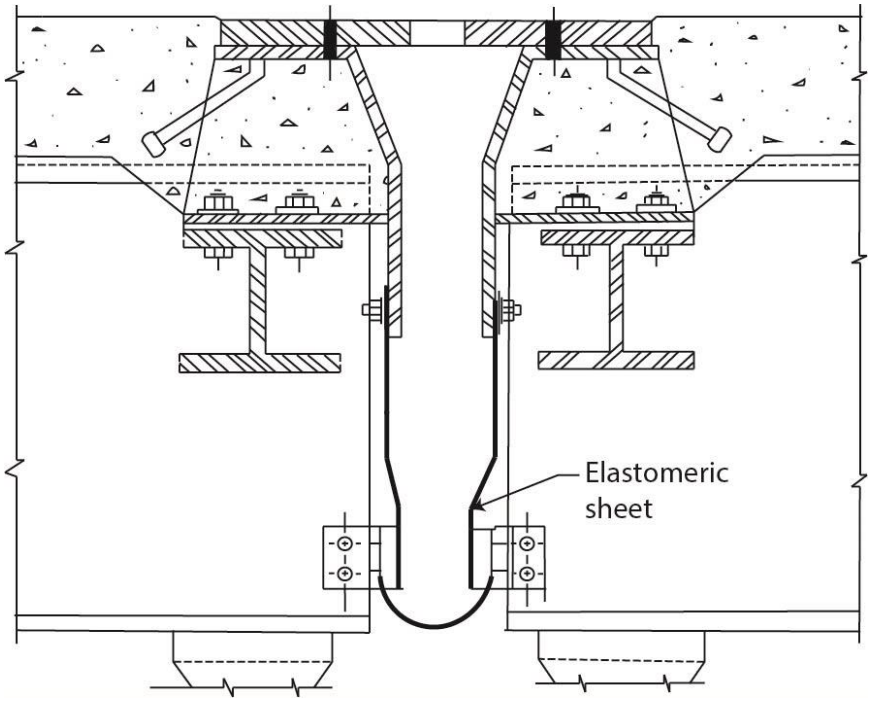

Figure 10.14 Drawing of a Drainage Trough

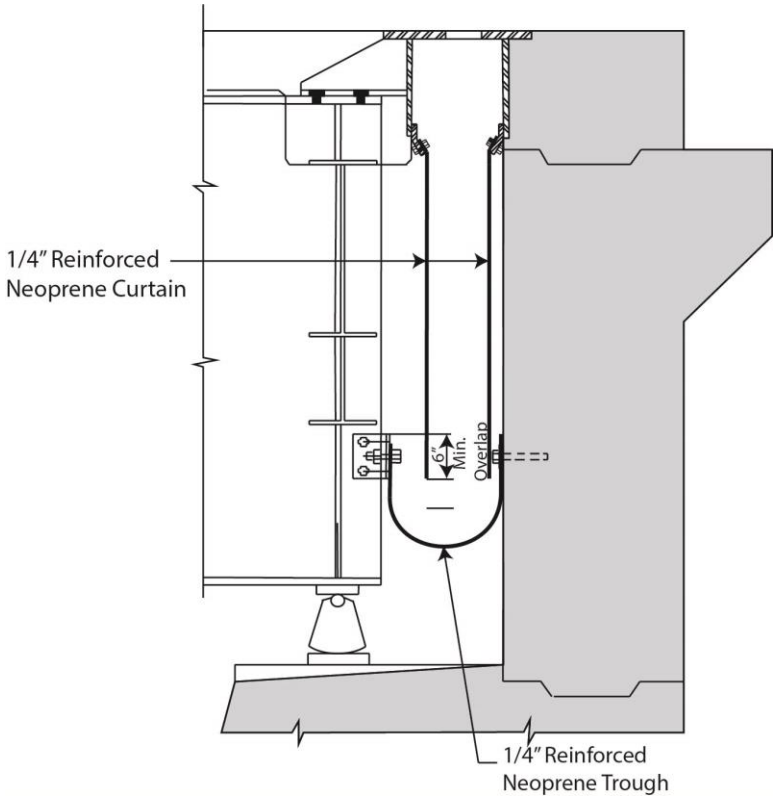

Figure 10.15 Drawing of Finger Joint with Elastomeric Trough

Figure 10.16 Elastomeric Trough

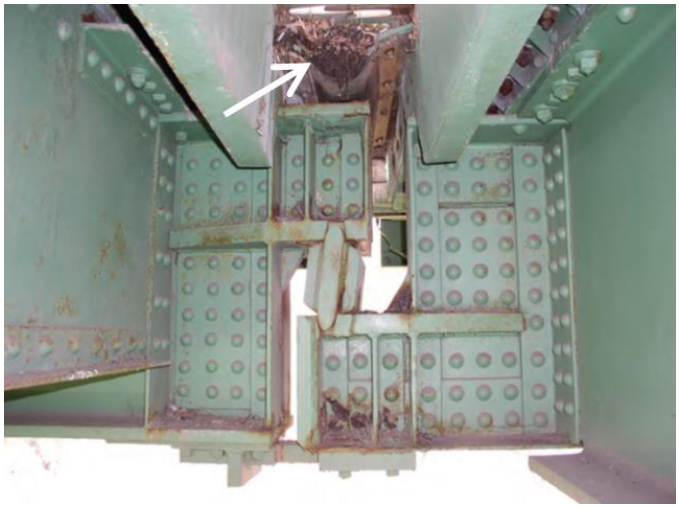

Figure 10.17 Clogged Elastomeric Trough

10.2 Preventive and Basic Maintenance of Deck Drainage

What To Look For

- Debris
- Clogged elements
- Evidence of ponding
- Leaks
- Broken frames
- Broken piping/fittings
- Loose hardware

Regardless of the bridge deck drain type, size, or shape, cleaning is required as part of Preventive Maintenance. Frequent sweeping and flushing of the bridge deck will remove sediment and debris which collects in the bridge shoulder and bridge inlets. The end result of frequent cleaning is less required bridge deck drainage system maintenance.

10.2.1 Cleaning

The necessary cleaning of bridge deck drain systems can be accomplished by several methods. These methods include:

- Shovel
- Rodding
- High pressure water jetting
- High pressure jetting with auger
- Plumber snake

These cleaning methods should be coordinated with proper traffic control above and below the bridge to ensure traffic is not affected by the cleaning operation and resulting water or debris. In addition, crews should be provided with proper personal protective equipment for cleaning operations and possess appropriate equipment to collect and properly dispose of the resulting slurry and debris from the cleaning operations. A typical procedure for cleaning bridge inlets and piping follows:

Suggested Procedure

Cleaning Bridge Inlets and Piping

1. Remove grating from inlet and remove debris from inlet base with shovel or vacuum truck. If grating is frozen, may need to remove some debris with high pressure water to free grate.
2. Use snake to remove clog or debris from pipe.
3. Depending on extent of clog use high pressure jet with auger to remove clog or debris from piping (Removal of clogging or debris should be done from cleanout sections where available).
4. Flush the inlet base and piping with high pressure water to fully clean the surfaces of the system.
5. Replace grating and cleanout plugs (where available).

Cleaning of elastomeric troughs typically only requires the use of high pressure water jetting from the deck level and thru the open deck joint. Depending on the design and construction of the joint, rodding of the debris combined with jetting may be required.

10.2.2 Flushing

Flushing of closed bridge drainage systems is typically accomplished by high pressure jetting with water or compressed air. For high pressure jetting with water, two basic nozzle styles are used for drain pipe cleaning applications. Rear jets provide propulsion of nozzle while cleaning the drain pipe wall surface. Forward jet is used to penetrate blockages in the drain pipe for the nozzle to pass and through cleaning by rear jets. Flushing may also be accomplished with compressed air. When flushing with compressed air, a plug is typically installed at the lower end of the drain pipe to make it airtight. The debris in the pipe is then saturated with water from the deck level. Next a small diameter pipe is inserted thru the plug and introduces high pressure

air which will loosen any obstruction in the drain pipe. Figure 10.18 shows nozzle jet heads used for flushing drain pipe.

Forward Jet nozzles used for cleaning clogged pipes

(a)

Rear Jet nozzles used for cleaning pipes that are not clogged

(b)

Figure 10.18 Jet Nozzles Used for Cleaning (a) Forward Nozzles for Clogged Pipes and (b) Rear Nozzles for Non Clogged Pipes (Courtesy of US Jetting, LLC.)

10.3 Repair and Rehabilitation of Deck Drainage

Deck drainage systems will require repair, rehabilitation, and possibly retrofit in order to restore functionality to proper collection and discharge of runoff from the bridge deck. There are several approaches to address maintenance repairs or retrofits to deck drainage systems. Any scupper or inlet penetration within the deck should have the perimeter sealed after the cleaning operation to ensure water and chlorides do not migrate along the interface between the scupper/inlet and the bridge deck. For decks with asphalt wearing surfaces, an asphalt cement sealer is adequate. For bare concrete decks, a methacrylate sealer is appropriate.

10.3.1 Redirecting Flow off Bridge Deck

One solution to addressing a poorly designed and unmaintainable bridge deck drainage system would be to redirect flow off the bridge deck to bridge end inlets, and may only be suitable for short to medium span bridges. This would require plugging of existing bridge scupper/inlets (deck or barrier penetrations) and construction of bridge end inlets near the ends of the bridge approach slabs. In addition, all bridge deck joints should be evaluated and repaired or replaced to ensure properly/effectively sealed to eliminate substantial runoff thru the deck joint systems. Prior to implementing this solution, a hydraulic analysis should be performed to confirm that the resulting spread with no bridge deck inlets was within allowable limits. Refer to Figure 10.9 for a typical bridge end inlet configuration.

10.3.2 Drain Extensions

On many older bridges, pipe scuppers were installed at the curb line to remove runoff from the bridge deck. The penetrations typically terminated at the bottom of the bridge deck which allowed the runoff to come directly into contact with the superstructure and substructure, resulting in accelerated corrosion of steel and spalling of concrete. Several agencies have developed drain extension retrofits to eliminate the direct exposure of superstructure and substructure elements to the deck runoff. If the penetration terminates at the underside of the bridge deck, these retrofits involve the installation of a new pipe (PVC or Fiberglass) from the deck level to a distance of about 3 inches below the superstructure member. Refer to Figure 10.19 and Figure 10.20 for photographs of PVC drain extension retrofits. The extension below the superstructure should be maximized to ensure that wind will not blow the runoff back onto the superstructure elements; 2-3 feet has been used by certain agencies and was found to be effective. If an existing downpipe has been provided but terminates just below the deck level, the retrofit would involve the extension of the downpipe so that it terminates below the superstructure member. Refer to Figure 10.21 for an example of a drain extension detail. Depending on the height of the downpipe above the ground and conditions below, consideration should be given to the installation of scour pads at the ground level to eliminate soil erosion. This may be accomplished with riprap stone and a filter fabric, or with a precast concrete pad.

Figure 10.19 PVC Drain Extension Retrofit at Fascia

Figure 10.20 PVC Drain Extension Retrofit Beneath Deck
(Courtesy of the New Jersey Turnpike Authority)

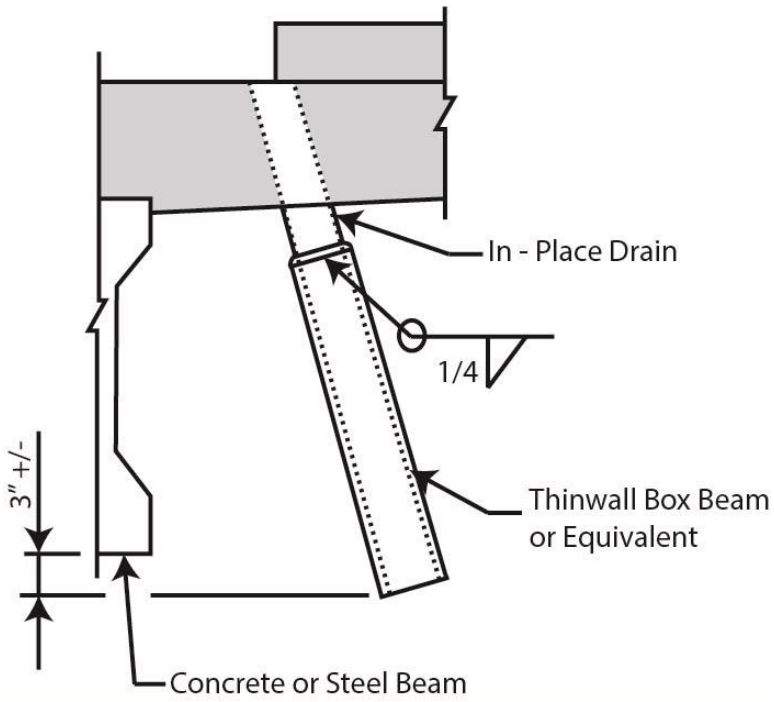

Figure 10.21 Steel Pipe Drain Extension

10.3.3 Inlet Grate Replacement

Scupper or inlet grates should be evaluated for grate opening size and effectiveness in blocking large debris from entering the bridge drainage system. Larger grate openings are a primary reason for inlet clogging. Replacement with filter grates with smaller openings will eliminate larger debris which ultimately allow for clogging of the inlet and piping system. As part of the grate replacement process, consideration should be given to cleaning the inlet frame and coating with a cold applied galvanizing material to provide a smoother surface. The coating will minimize small debris and sediment collection on shallow frame surfaces. Figure 10.22 shows an inlet with large grate openings filled with debris. The drawing in Figure 10.23 provides retrofit filter grate details. Installation photos are shown in Figure 10.24 and Figure 10.25.

Figure 10.22 Bridge Inlet Grate Which Permits Clogging of Inlet
(Courtesy of the New Jersey Turnpike Authority)

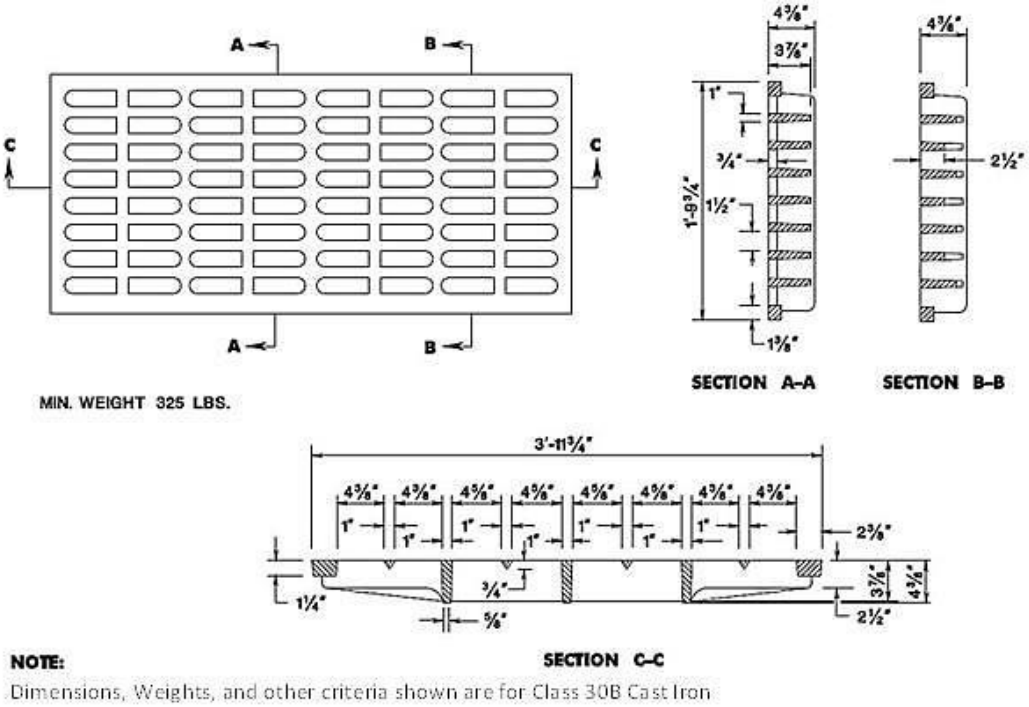

Figure 10.23 Drawing for a Bicycle Safe Grate (Cast Iron)
(Courtesy of the New Jersey Department of Transportation)

Figure 10.24 Bridge Inlet Grate Retrofit (Filter Grate)
(Courtesy of the New Jersey Turnpike Authority)

Figure 10.25 Bridge Approach Inlet Grate Retrofit (Filter Grate)
(Courtesy of the New Jersey Turnpike Authority)

10.3.4 Piping Replacement

Many closed bridge drainage systems were built with small diameter piping, using either orthogonal (90 degree bends), or small radius bends, with flat slopes, limited or no cleanouts, and drain hopper systems. These systems as designed and constructed are prone to severe clogging. Maintenance retrofits to these systems could include replacement of orthogonal piping, or small radius bends, with large radius bends to improve water flow and decrease clogging, installation of cleanouts to allow for proper maintenance, increasing the slope of the drain pipe, elimination of drainage hoppers, and if necessary replacement with larger diameter pipe (6 inches minimum or larger). Many agencies are now using fiberglass materials for maintenance drainage retrofits as they are much lighter and easier to install given weight reduction over steel pipe, and the materials are not subject to corrosion. Refer to Figure 10.26

which shows a closed bridge drain system with orthogonal piping, smaller diameter pipe with flat slopes, and drainage hoppers. Compare this to Figure 10.27, which is a newer fiberglass bridge drain system with larger diameter pipe with steeper slopes, long radius bends for transitions, and easily accessible cleanouts for maintenance.

Figure 10.26 Closed Bridge Drainage with Orthogonal Piping Prone to Clogging (Courtesy of the New Jersey Turnpike Authority)

Figure 10.27 Closed Bridge Drainage System with Steeper Pipe Slopes, Long Radius Bends and Cleanouts (Courtesy of FRPbridgedrainpipe.com)

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Chapter 11 - Superstructure

The superstructure consists of the structural framing system supporting the deck, which can include beams, diaphragms, trusses, floorbeam and stringer arrangements, to name a few. Although the deck is designed to carry vehicular loading it also serves as a protective cover for the framing, bearings and substructure. The deck joints are also critical in protecting the bearings, framing members, and substructure from exposure to the corrosive salt-laden runoff of the roadway.

11.1 Preventive and Basic Maintenance of Concrete Superstructures

There are many different types of concrete superstructures, including:

- Concrete slabs (solid and voided slabs)
- Concrete frames, concrete arches
- Concrete Tee beams or T-beams (Bulb Tee, Double Tee, Quad Tee and Rib Tee)
- Prestressed concrete I-girders
- Prestressed concrete slabs and box beams/box girders (single, spread or multiple/adjacent)

Preventing concrete deterioration is much easier and more economical than repairing deteriorated concrete. This prevention begins in the design of the structure with the selection of the proper materials, detailing, mixture proportions, concrete placement, and curing procedures. Even a well-designed concrete superstructure will generally require follow-up maintenance action.

Below are some examples of design and construction considerations that improve or affect the durability of superstructures:

- Precast prestressed structures – manufactured assembly line style in a large yard, the fast pace of girder production ignores some basic concepts of concrete preventive maintenance such as concrete cover. The ends of prestressed concrete girders usually have exposed high strength steel strands which can quickly corrode. Some bridge owners require the strands to be exposed, cut and covered with mortar. Many do not require any modification. Figure 11.1 shows a precast beam in place seated on an abutment with potentially inadequate cover for strands. Although the figure shows exposed strands, this may not necessarily be a problem. For example, if the end of the girder is encased in concrete, as with an integral abutment, the strand ends are only exposed for a short period of time until the abutment is poured.

Figure 11.1 Exposed Strands on End of Precast Prestressed Concrete Girder

- Precoating of girders – Applying surface protection to critical areas of the concrete girder such as its ends can extend the service life of the girder. Researchers in Wisconsin have found the most effective way of preventing beam end corrosion was to apply a polymer resin coating to the beam ends before installing them in the field.
- Drainage – design of drainage that will prohibit water from leaking onto the concrete; for example, extending drain pipes 6 inches beyond the lower face of the superstructure.
- Precast box girders – This particular structure type can be problematic from a maintenance standpoint. Construction of precast prestressed concrete placed directly adjacent to one another can be troublesome to inspect and maintain. Maintenance issues for this type of superstructure include:
    - Clogged drains leading to water in the voided cells of the box
    - Shear key failure between adjacent box beams leading to overload and reflective cracking into the bridge deck
    - Exposed strands on beam ends corroding quickly
    - See Section 11.4.7 for additional discussion on precast box girders

  As an example of how details and specifications play such an important role in future bridge maintenance, there are many precast box girders that have been in service for 25 years that have not experienced any of the problems noted above. Poured in place concrete decks over the box girders have eliminated the shear key failure issues, and recent construction specifications have called for exposed strands to be coated and covered.

The primary types of maintenance for concrete are:

- Cleaning (Section 11.1.1)
- Surface protection (Section 11.1.3)
- Joint restoration (Chapter 8)
- Minor concrete repair (Section 11.1.4)

The approach to preventive maintenance for concrete superstructures varies from owner to owner. There are owners that clean and wash their bridges regularly, but many do not. Some bridge owners mandate a surface protection be applied to the concrete during initial

construction, some never require it. However, it is generally recognized that a clean and protected concrete surface is beneficial.

What To Look For

In providing maintenance to concrete superstructures, it is always important to look out for a superstructure that may be in distress. This could include:

- Overheight vehicular impact damage
- Large cracks greater than 1/8 inch in reinforced concrete
- Spalls that expose reinforcement

11.1.1 Cleaning

Cleaning of concrete superstructures is extremely important. Regular cleaning of the superstructure members is necessary to remove accumulation of sand, debris, bird droppings, and other harmful material by flushing with high-pressure water jet or compressed air, sweeping, or shoveling to remove build up. This is particularly critical in areas where there are likely to be chlorides in the debris or on surfaces, such as areas where salt is used for snow and ice control, and marine environments.

In addition to cleaning for concrete surfaces, ancillary elements such as drains should be cleared to allow proper drainage off the structure. An example of preventive maintenance as applied to drainage is the extension of vertical drain pipes to 6 inches beyond the lower flange of the concrete girder (See Chapter 10).

11.1.1.1 Debris Removal

Traffic, pedestrians, animals, and/or flooding cause superstructure debris. All debris should be removed for safety reasons. Additionally, debris should be removed to prevent deterioration in areas where debris accumulates and can trap moisture such as on flanges and diaphragms. This debris could be considered hazardous and prior to removal and disposal local environmental protection requirements should be checked.

11.1.1.2 Power Washing

Power washing the concrete superstructure removes the salts that have accumulated and/or adhered to the superstructure. Areas include sides of beams and diaphragms exposed to spray or where water has come off the deck, beam ends and seats under deck joints, and at deck joints or drains that leak or allow water through. Other areas could include sides of beams adjacent to deck drains.

11.1.2 Bird Control

If bird nesting or congregation is apparent, these areas should be cleaned and screens, spikes or other deterrents installed, as shown in Figure 11.2. Here a simple rubber cylinder was installed to fill in the gap between bridge bearings where a bird can build a nest. There are so many forms of bird habitat controls available, they could not be covered in this manual. Past experience or internet searches are suggested to obtain the right solution for a particular

bridge. Caution should be exercised as there may be restrictions due to federal protection of birds or other species such as bats.

Figure 11.2 Evidence of Bird Nesting (left) and a Rubber Cylinder Deterrent (right)

11.1.3 Concrete Superstructure Protection

Concrete superstructures are shielded from much of the traffic and weather by the bridge deck. The areas of concrete superstructures that can require protection are areas below deck joints, around deck drainage components, and on exterior girders. The noted superstructure concrete areas are exposed to spray or water flowing from the bridge deck and are thus susceptible to chloride exposure and the potential for freeze thaw damage.

The most common treatment to protect the concrete from water and chlorides is to apply spray on waterproofing products. Waterproofing products fall into two general classes: pore blockers and hydrophobic (water repelling) products. Pore blockers typically used are epoxies and silanes or siloxanes. Epoxy materials are also popular to treat beam ends under joints.

While the entire superstructure can be coated with protection, the application is often directed to critical areas such as beam ends.

Suggested Procedure

Spray-Application of Concrete Protection

1. Clean the area to be treated with high pressure water or light sand blasting.
2. Use compressed air to remove all loose dust or debris when dry.
3. Apply the waterproofing products according to the manufacturer recommended method (typically spray applied using low pressure sprayers). Spray evenly on dry concrete starting at the lowest point and working up.

An effective method of preventing beam end corrosion is to apply a polymer resin coating to beam ends before installing them in the field. Epoxy coating is the most commonly used

polymer. An alternative treatment is to apply a penetrating sealer to the concrete end beams. Use of a Fiber Reinforced Polymer (FRP) wrap can be about equally effective, but is more costly and difficult to install than the polymer coating and is usually only used as a repair measure.

11.1.4 Spall and Crack Repair

Preventive and basic maintenance of concrete superstructures focuses on preventing the deterioration of reinforcing steel. Minor cracking, scaling and spalling can exacerbate the corrosion of rebar by allowing corrosive water and salt to infiltrate the concrete. Cracks or spalls large enough to affect the structural design or make the bridge unsafe for vehicular or pedestrian traffic will be covered in Section 11.4 Maintenance and Repair of Concrete Superstructures.

Minor cracking can often be repaired with sealants or by other penetrating materials such as silane, methacrylate, or epoxy. Flexible joint sealants, epoxies and cement grouts are the most common maintenance for routed cracks, which is a crack that is intentionally widened near the surface, while epoxy injection is also an option for cracks as narrow as 0.002 inches, as shown in Figure 11.3 through Figure 11.4.

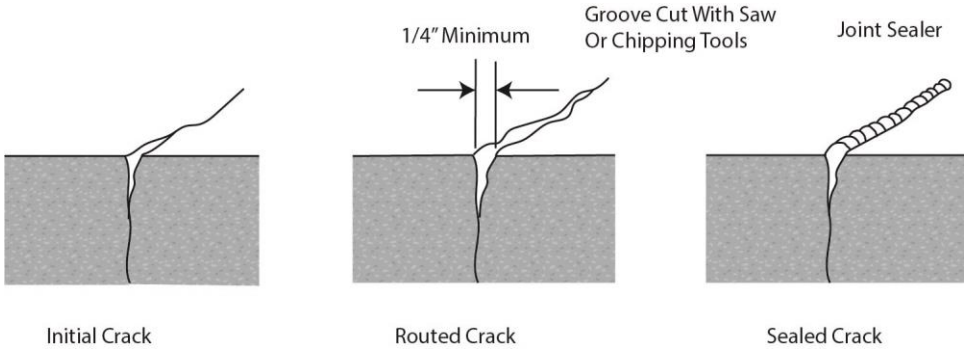


Figure 11.3 Conventional Procedure for Sealing Cracks

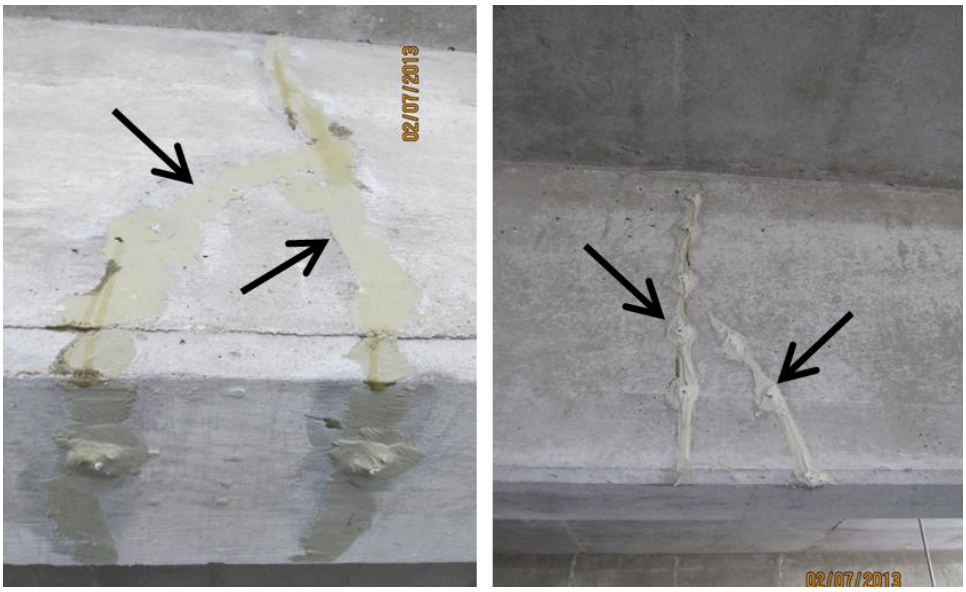


Figure 11.4 Epoxy Injected Cracks on a Superstructure

To prevent minor scaling and spalling from growing into a major repair, it is best to remove unsound concrete as soon as possible and replace with a concrete type material that will protect the steel reinforcement while being structurally adequate. Options may include a simple mortar patch for small areas to use of shotcrete for larger areas. Deeper spalls and repairs requiring formwork and replacement concrete are discussed later in Section 11.4 Maintenance and Repair of Concrete Superstructures.

In summary, preventive maintenance of concrete superstructures includes:

1. Keeping the concrete surfaces free of salt, dirt and debris by regular cleaning
2. Control and prevention of bird nesting and congregation which leads to deposits of corrosive waste
3. Inspecting for and repair of scaling, minor spalls, delaminations and cracks as soon as possible
4. Sealing the concrete to prevent chloride intrusion

11.2 Preventive and Basic Maintenance of Steel Superstructures

Preventive maintenance of steel superstructures consists mainly of measures to clean to steel and protect the steel from corrosion. The preservation of steel involves protection from exposure to electrolytes, such as water or soil. When deicing salt is added to the electrolyte, there is a dramatic increase in the rate of corrosion of the structural steel.

Common protective coatings for steel superstructures are weathering steel, galvanizing, metalizing, and paint. Weathering steel is a type of steel that forms its own protective coating and theoretically does not need painting. However, there have been cases of poor performance of bridges constructed with this type of steel. Therefore, members constructed from weathering steel should be monitored for excessive corrosion and be painted if necessary. Typical painting requirements are based on whether the steel is new or is to be repainted.

What To Look For

When performing maintenance on steel superstructures it is always good to see if anything does not look right. This includes:

- Impact damage to steel fascia girders. Generally, the girder being twisted and bent from impact
- Severe corrosion and section loss
- Bent cross frame members indicating movement and buckling

11.2.1 Cleaning

Similar to concrete superstructures, the cleaning of steel superstructures is extremely important. Regular cleaning of the superstructure members is necessary to remove accumulation of sand, debris, bird droppings, and other harmful material by flushing with high-pressure water jet or compressed air, sweeping, or shoveling to remove build up. This is particularly critical in areas where there are likely to be chlorides in the debris or on surfaces, such as areas where salt is used for snow and ice control, and marine environments.

Part of the cleaning process should be the clearing of any drains and downspouts that could clog and inadvertently flood steel components. A simple modification such as extending the superstructure drain pipe 6 inches below the bottom flange of the girder is considered a preventive maintenance task.

11.2.1.1 Debris Removal

Traffic, pedestrians, animals and/or flooding create superstructure debris. All debris should be removed for safety reasons and to prevent deterioration in areas where the debris accumulates and could trap moisture such as on flanges, diaphragms, gusset plates, etc. Steel bridges tend to collect debris more than concrete due to the complicated connection details and horizontal elements such as bottom flanges of girders.

11.2.1.2 Power Washing

Power washing the steel superstructure removes the salts and debris that have accumulated and/or adhered to the superstructure. Areas include sides of beams and diaphragms where water has come off the deck and at beam ends, under deck joints or drains that leak or allow water through. Other areas could include sides of beams adjacent to deck drains. A difference in the use of power washing for steel versus concrete superstructures is that every steel superstructure has some kind of protective coating that may be sensitive to very high pressures. Care should be exercised.

11.2.2 Bird Control

Bird waste can be quite corrosive, and the accumulation of nesting materials can prevent the proper draining of rainwater from the bridge, resulting in ponding which promotes corrosion. Steel bridges give birds plenty of areas to nest and stay out of the weather. In particular, steel bridges with wide flanges or gusset plates are attractive to birds, an example of which is shown in Figure 11.5. Trusses and towers also make particularly good nesting areas. Wash these areas and install bird screens or other deterrents routinely. As with concrete superstructures, maintenance workers should be aware of any environmental restrictions on federally protected wildlife.

Figure 11.5 Bird Nesting Signs on Steel Superstructure (left) and Behind Steel Girder (right)

11.2.3 Painting

Painting of steel superstructures should be performed on a regular basis with the repainting schedules based on the type of coating used, the environment, and the rate of coating deterioration. This routine maintenance prevents corrosion and section loss of the steel members. Painting operations require properly designed and approved containment systems. An example of a painting containment system is shown in Figure 11.6.

Figure 11.6 Painting Project with Approved Containment System

Less costly painting operations include:

- Spot Painting – Painting of localized areas on the steel superstructures that have lost their protective coating or areas of rust and corrosion. Once steel members begin to corrode, they begin to lose strength due to material loss. It is important that areas of spot rust be touched up routinely. The loose rust must always be removed before the touch up. Importantly, if the corrosion is due to exposure from a leaking drain or joint, maintenance or repair of the source defect must be coordinated with the spot painting. It is also critical to follow the coating manufacturer's recommendations for applying the coatings (temperature, humidity, and application method) to maximize the life of the coating.
- End of Beam Painting – Many agencies recommend painting / repainting superstructure steel within a distance of 1.5 times the depth of the girders from the bridge joints for extra protection. If aesthetics are a concern, the exposed areas of the fascia girders are painted for the entire girder length.

11.2.4 Weld Peening

Most modern structures use welding to connect steel plates. For the weld to be successful, the adjoining plates are brought up to a molten state as the weld material is added. Once everything cools down, there are built up stresses in the weld, as the weld material tends to shrink. This locked in tension stress is one of the reasons welds can crack. A unique idea is to apply a compressive force to the weld after the bridge is erected to counteract the tension. Imagine a hammer hitting the weld to compress it thereby relieving these built in stresses. That is the basic concept of peening.

Figure 11.7 shows a tool that uses oscillating impact pins at a very high frequency to compress the weld. This is usually performed by a trained technician. For this technology to work, the structure has to be in its final position, so if a bridge girder would need to be erected.

When to Call the Engineer

Call the Engineer before performing weld peening operations, as weld peening is not applicable in all situations.

Figure 11.7 Peening of Weld using Ultrasonic Impact Technology

The following references provide more information on peening:

- *NHI 134062 Bridge Rehabilitation and Design*, U.S. Department of Transportation, Federal Highway Administration
- http://digital.lib.lehigh.edu/fritz/pdf/417_2_77_.pdf
- http://www.fhwa.dot.gov/bridge/steel/pubs/hif13020/hif13020.pdf

In summary, preventive maintenance of steel superstructures includes:

- Keeping the steel surfaces free of salt, dirt and debris by regular cleaning
- Control and prevention of bird nesting and congregation which leads to deposits of corrosive waste
- Cleaning rust, painting and/or spot painting as needed
- Possible peening of welds
- Ensuring drainage downspouts extend at least 6 inches below the bottom flange of the girder

11.3 Preventive and Basic Maintenance of Timber Superstructures

There are many different types of timber superstructures, including:

- Log Beam
- Sawn Lumber
- Glulam Concrete slabs

What To Look For

When performing maintenance on timber superstructures, it is a good idea to look out for things that may cause structural distress to the bridge. This includes:

- Look out for large splits or cracks in the timber bridge elements
- Look out for broken or crushed timber
- Look out for excessive displacement of the timber girders indicating a possible overload
- Look out for excessive decay and insect damage

Preventing timber deterioration is much easier and more economical than repairing deteriorated timber. This prevention begins in the design of the structure with the selection of proper materials, preservatives, and other means to protect the timber from moisture.

11.3.1 Preventive Maintenance and Cleaning

Mechanical damage, insect attack and deterioration from decay are potential threats to even the most durable timber superstructures. Preventive and maintenance measures may be taken to keep structural members in good condition to reduce future problems. This would include regular cleaning.

11.3.2 Moisture Control

The most economical and simplest method of preventive maintenance is moisture control. Moisture contents within timber members of greater than 20 percent support fungal and insect growth. When exposure to wetness is reduced, fungal and insect growth is inhibited. A maintenance worker may utilize a moisture meter to measure moisture contents.

11.3.2.1 Identify Visible Area

Visibly high moisture content areas may be identified by locating the source of water and taking corrective action to eliminate the source.

11.3.2.2 Controlling Drainage on Timber Superstructures

Restricting water passage through the deck substantially reduces the potential of decay to structural members below. Bituminous roofing cement can be used to help seal all types of timber decks.

Lumber running planks used as a wearing surface are often subjected to water ponding. Installation of tubes through the deck can be used to drain water down and away from the deck and supporting members, as shown in Figure 11.8.

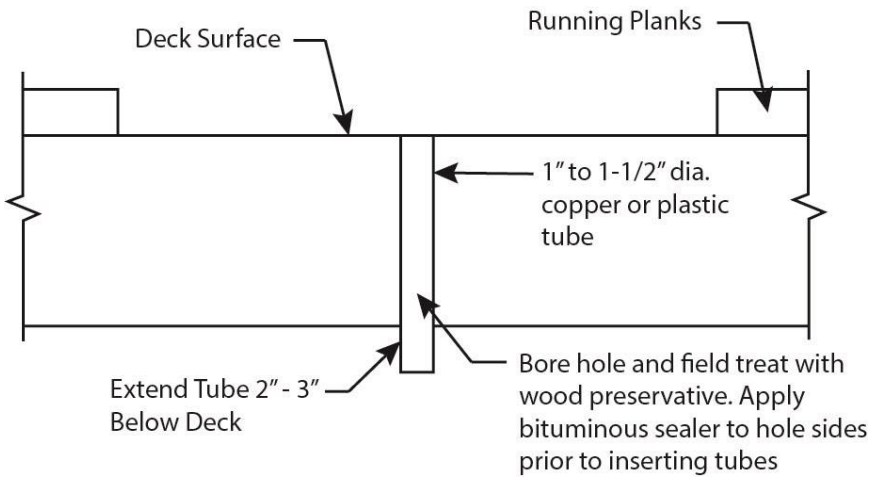

Deck Surface
Running Planks
1″ to 1-1/2″ dia. copper or plastic tube
Extend Tube 2″ - 3″ Below Deck
Bore hole and field treat with wood preservative. Apply bituminous sealer to hole sides prior to inserting tubes

Figure 11.8 Installation of tubes through deck to drain water away from superstructure elements

11.3.3 In-Place Preservative Treatment

The application of a suitable preservative treatment will increase the longevity of a timber structure by reducing the effect of weathering, drying out, and surface rotting. In place preservative treatment is most effective when applied before decay begins and may be used to treat splits, checks, and delaminations. Additional surface treatment information, including surface treatment of new materials and pressure treatment, is included in Section 13.3.3 Preservation Treatments for Substructures.

11.3.3.1 Surface Treatments

Brushing or spraying the preservative in liquid form is the simplest and least expensive treatment. The wood surface should be thoroughly saturated in order for the preservative to be fully effective. It is preferable to apply preservative to dry wood. Wet wood absorbs less preservative than dry wood. If due to environmental issues, preservatives must be applied to wet wood, double the amount of preservative should be applied. Preservatives are available in oil based or water based solutions.

A commonly used oil base preservative for brush or spray applications is copper naphthenate. It is available as a concentrate or in ready–to-use liquid in drum or gallon containers. The use of bituminous or asphaltic pastes could be used as effective sealants.

Supplemental treatments for timber bridges have recently been boron based. Boron is a chemical element that has been effective against both decay fungi and insects and is not very toxic to humans and other mammals. Other advantages of borates are:

- Odorless
- Colorless
- Does not interfere with finishes
- Corrosion inhibitor
- Can be applied in multiple ways such as a powder, gel, paste or rod

Borate solutions are water-based and may be sprayed in checks or splits. However, borate preservatives may leach from the wood during precipitation. Borate solutions are sold as a concentrated liquid, powder, gel or rods.

Borate is effective when the wood is moist; at least 20 percent moisture content is needed to achieve diffusion, which is the movement of the chemical through the wood. At 40 percent moisture borate diffusion is very rapid. Thickened glycol-borate solutions are a syrupy type liquid that allows for a greater concentration of the chemical element than powder. The solution can be applied by spray or brush, or used to flood cut-ends or holes. Borate gels are available in tubes and can be applied by standard caulking guns. An advantage of the gel formulation is that it can be applied to voids, cracks, and any kind of holes. Borate gels are the most convenient but also most expensive. Lastly, a paste formulation of borate typically contains at least one component of borate that diffuses into the wood and at least one other that can provide long-term protection near the application. This method is popular for application to piles at the ground line.

Some preservative treatments besides borate are also available in semisolid greases or pastes which may be effectively applied to vertical surfaces. These commonly contain sodium fluoride, creosote, or pentachlorophenol.

In Figure 11.9, the stringer exhibits splitting and checking at the bottom. As long as the capacity of the beam is not compromised, it may be beneficial remove the deteriorated section, and to treat these stringers with a paste or semisolid grease to prevent further deterioration. It is recommended to apply these preservative treatments every 3 to 5 years.

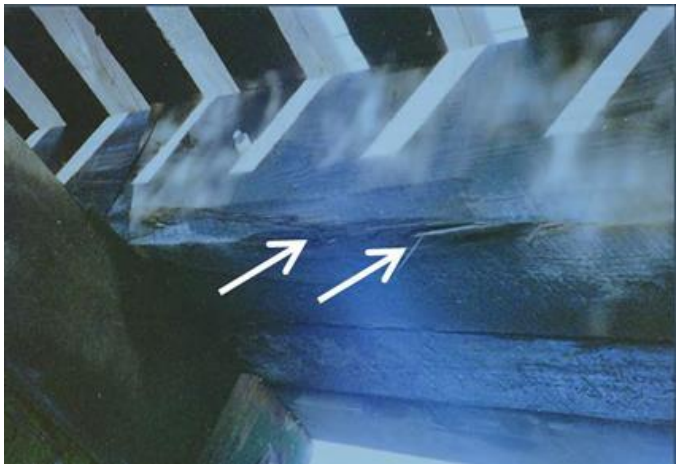


Figure 11.9 Splitting and Checking

11.3.3.2 Internal Treatments

Fumigants are gases used to internally treat large piles or timbers. Fumigant preservative chemicals are available in liquid or solid form that can be used to control internal decay from fungi and insects. The fumigants are placed in pre-bored holes. Over time, fumigant preservatives transform into gases which spread through the wood as much as 4 feet in horizontal members such as beams.

The pattern of bores may be determined by the condition of the wood. Fasteners, seasoning checks, badly decayed wood, and other openings to the atmosphere should be avoided.

Holes for application of fumigants in horizontal members such as beams should not be more than 4 feet apart. Additional information and recommended dosages for fumigants may be obtained from the chemical manufacturers.

Common Liquid fumigants used are Vapam (33-percent sodium N-methyldithiocarbamate), Vorlex (20-percent methylisothiocyanate, 80-percent chlorinated C3 hydrocarbons), and chloropicrin (trichloro-nitromethane). Liquid fumigants may be applied by using commercial equipment or may be applied from 1-pint polyethylene squeeze bottles. Liquid fumigants should not be stored in plastic bottles for long periods because the plastic can become brittle and crack. Leaks may sometimes occur during application of the fumigant, in which case the bore hole should be plugged and be moved to an alternate sound wood location.

Solid fumigants are typically composed of methylisothiocyanate (MIT), which is the active ingredient of Vorlex. Solid fumigants are inserted directly into the pre-bored holes. Holes should be placed on both sides of checks or splits and bored to within 1 ½ to 2 inches of the bottom of the member. The hole is plugged shortly after placing the chemicals with a treated-wood dowel driven tightly into the wood. This procedure is shown in Figure 11.10. A 10-year treatment cycle is recommended. When inspections indicate the presence of active decay, retreatment may be required sooner than the 10 year cycle. It is important to keep accurate records including the date and location of the application, the type of chemical, and the dose. A metal tag on the member noting treatment information may be placed. However, these tags should not be the sole means of recording treatment information, in case they are removed in between applications. The same holes may be used for retreatment, and the holes may be replugged.

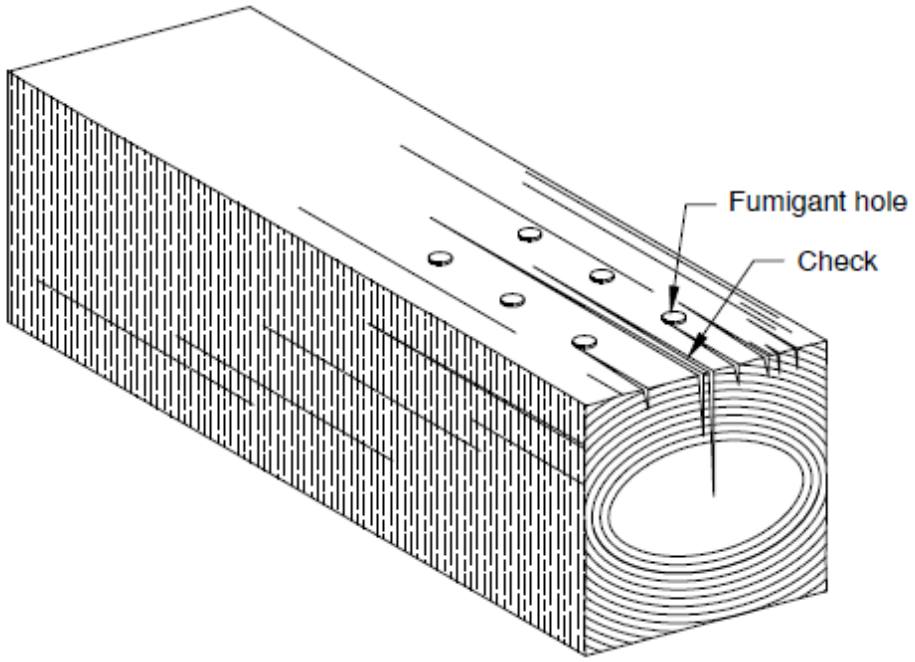


Figure 11.10 Solid Fumigant Inserted Directly into Pre-Bored Holes

For in-place application of fumigants, it is important to select an experienced contractor who fully understands their use and the required safeguards. The treatments are toxic to humans and must be used in accordance with State and Federal laws.

The treatments pose minimal environmental or health hazard when proper controls and precautions are applied. The environmental damage potential can be higher in some field locations due to the location, environment and distance from water sources. A gas mask with the appropriate filter available for emergency use should be available for fumigant applicators. When experiencing strong odor or eye irritation during fumigant application, all individuals should move upwind from the treating area and allow vapors to clear. When any form of in-place treatment is used, the procedures, precautions, and contingency for accidental spillage or injury should be well planned before beginning treatment. In-place treating by local maintenance crews is limited by the scope of the treatment required. For routine maintenance, the amount of treating required is normally minor, and local crews can be used when properly trained personnel are available. For larger projects involving many members or an entire structure, it is advisable to contract the project to specialists in the field. There are companies that have safely provided in-place treating services for many years.

Figure 11.11 shows examples of borate rods which range in size from 0.75 inch to 0.5 inch diameter.

Figure 11.11 Borate Rods

Borate rods contain compressed chemicals fused into a solid form produced by heating and molding. The advantage of the rod formulation is their ease of application and low risk of spillage. They can be applied to holes drilled upward from below a member. A disadvantage of the borate rod is that no water is applied, which is how the borate gets diffused into the timber.

11.3.4 Epoxy Repair

A major maintenance cost associated with timber construction is the repair of structural members near bolted joint connections which exhibit checking, cracking, and splitting. The use of pressure injected epoxy to repair damaged timber connections is an alternative which may be considered. Epoxy may also be manually applied as a gel or putty.

Suggested Procedure

Basic Epoxy Injection

1. Prepare member by cleaning
2. Determine areas for port injection and sealing
3. Inject the epoxy
4. Finish the surface, consider aesthetics

For manual application, port setting and joint sealing are not required.

Epoxy may be used as a grout to fill decay void, cavities, checks or splits to prevent water and other debris from entering. It forms a bond which aids in reducing further splitting and in preventing water and debris from entering the affected area. For surface repairs, voids may be filled with an epoxy gel. Liquid epoxy may be injected into a member exhibiting splits or insect damage.

When to Call the Engineer

An engineer should be consulted when removing large sections of the beam.

When the defect in the original member is the result of decay, the damaged wood must be removed to prevent further infection. If areas to be repaired show early signs of decay, in-place treatment may be adequate to help prevent decay, as long as the member strength is not compromised.

When removing decayed parts of members, it is recommended to remove visible decay plus an additional 2 feet of the adjacent wood in the grain direction. The moisture source should be identified and eliminated, if possible. When moist wood (greater that 20 percent moisture content) is found, the member should be dried before repairs are made. Although there are epoxies that will bond to moist wood, the presence of high moisture may provide conditions for continued fungal growth and deterioration. Surfaces must be thoroughly cleaned of all dirt and debris so that a good bond can be achieved between the wood and the epoxy.

To repair interior splits or voids in timber superstructures, epoxy can also be applied by pressure injection to reach these difficult areas. Ports must be installed and completely sealed before epoxy placement. The injection ports are holes bored into the timber member permitting epoxy injection into the member's interior. A port vent allows displaced air to escape as epoxy fills the void. The epoxy is distributed throughout the affected area when it exits the port vent. The minimum number of these ¼ inch to ¾ inch diameter ports is two. However, more may be required depending on the size of the repair area.

11.3.5 Stitching and Clamping

On timber-bridge trusses or other structures with a high number of small members, clamping and stitching steel are maintenance operations that may be utilized to prevent further

development of cracks, splits, or delaminations. The objective is not to close the split or check, but rather to prevent it from further developing.

Clamping and stitching are similar in concept in that both compress the component in question. However, clamping uses bolts and two-plate assemblies whereas stitching uses bolts or lag screws through one side of the member. Stitching may also use a plate for added strength. Schematics of each are presented Figure 11.12. Although both methods have been used effectively, clamping with bolts and steel plates is generally preferable because the section of the member is not reduced.

Caution should be exercised installing metal components in high humidity environments where there is cyclical temperature change. Condensation may form on plate assemblies and bolts exacerbating moisture related problems in the wood.

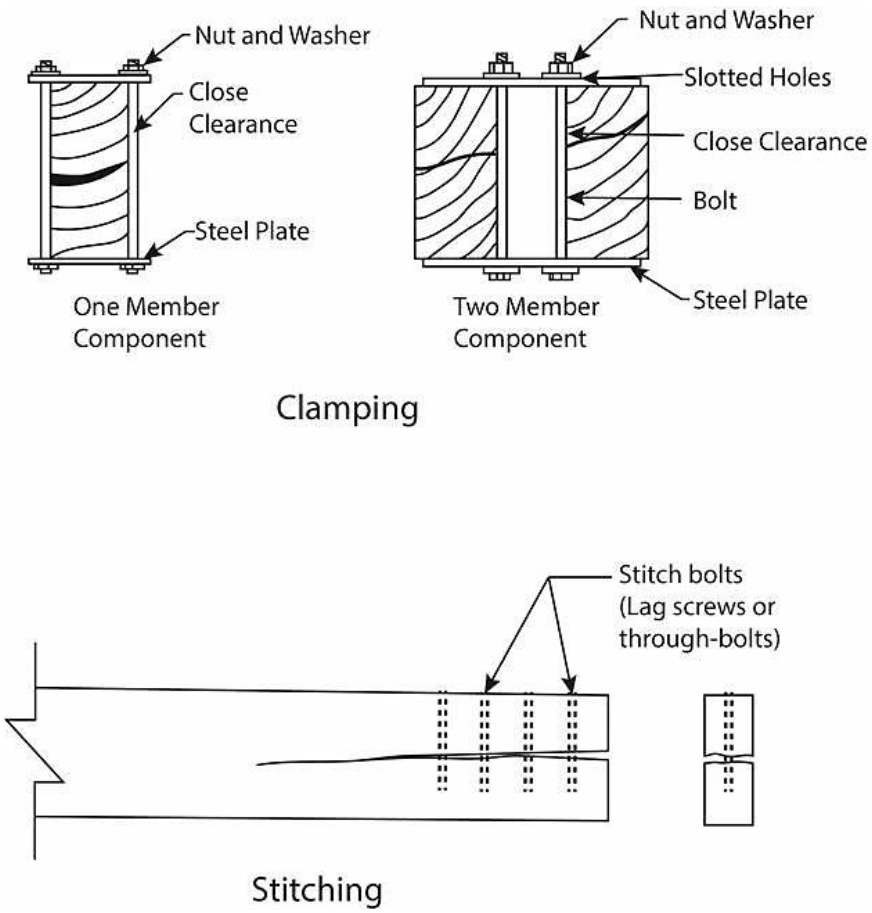


Figure 11.12 Typical Clamping (top) and Stitching (bottom) Details

11.4 Maintenance and Repair of Concrete Superstructures

What To Look For

- Spalls
- Cracking
- Efflorescence
- Leaking water
- Soft, unsound, or "punky" concrete
- Exposed reinforcing steel
- Section loss in reinforcing steel
- Exposed tensioning strands
- Damaged or broken tensioning strands

11.4.1 Introduction

The most common material for superstructure elements is concrete (reinforced or prestressed). Concrete is heavily used for bridge superstructures because it can be molded into almost any shape and has above average durability. Concrete superstructures may be precast in a plant or cast at the bridge site. Regardless of the type of concrete, the repair methods for the concrete material are similar.

All concrete superstructure elements use reinforcing steel to carry tensile and shear loads and limit cracking. Concrete superstructure elements typically use conventional reinforcing (rebar) or prestressing steel. The repair methods for conventional reinforcing and prestressed steel are different, which will be addressed in subsequent sections in this chapter. The main difference is that conventional reinforcing is passive and prestressed steel is active. A good analogy is a rubber band that is unstretched versus stretched.

This section presents common maintenance and repair techniques for concrete bridge superstructure elements.

11.4.2 Concrete Superstructure Patching

Concrete superstructure elements can be repaired using a number of techniques. The selection among the repair methods is dictated by the depth and extent of the concrete deterioration or damage. Concrete superstructure repairs are often required because of corrosion of reinforcing steel within the member, from traffic impact, or from freeze thaw damage. The repair of the concrete generally involves removing the deteriorated or damaged concrete and restoring the member with new concrete. Areas of concrete removal can be repaired in five different ways:

- Patching with trowel-applied or poured mortar
- Recasting with new concrete
- Prepacking dry aggregates and grouting
- Shotcrete
- Self-consolidating concrete

These methods of concrete removal are described in the following sections.

11.4.2.1 Patching

Patching small concrete spall areas with cementitious or resin mortars is the simplest of all concrete superstructure repair techniques. Concrete spalls between ½ and 2 inches deep can be repaired by patching without the use of formwork. An example of patching a concrete superstructure is shown in Figure 11.13. Note that overhead shallow repairs should be avoided, particularly those that do not engage existing rebar or aren't mechanically anchored. Examples of these situations include areas over traffic, over parking, and over pedestrians.

Figure 11.13 Patching of Concrete Superstructure (Courtesy of the New York State DOT)

Figure 11.14 shows the lower portion of a concrete superstructure ready for patching. The white blocks are cathodic protection elements. Cathodic protection is covered in Section 13.1.6 of this manual. The wood forms are in place in Figure 11.15. The final repaired girder section with forms removed is shown in Figure 11.16.

Figure 11.14 Concrete Superstructure Ready for Patch (Courtesy of Washington DOT)

Figure 11.15 Concrete Superstructure with Wood Form (Courtesy of Washington DOT)

Figure 11.16 Completed Repair of Concrete Superstructure (Courtesy of Washington DOT)

Suggested Procedure

Concrete Patch for Superstructure Members

1. Remove any loose concrete in the area to be patched.
2. Sawcut the perimeter of the removed concrete region in straight lines to a depth of ¾ inch to make a clean patch line. If possible, bevel the sawcut at 45 degrees inward to lock the patch in place.
3. Clean the surface of the existing concrete using hand tools, sand blasting and compressed air.
4. Thoroughly wet the concrete surface and allow it to dry on the surface.
5. Mix the patch mortar in accordance with the manufacturer recommended specification for vertical or inverted patches.
6. Apply a bonding agent to the existing surface, but note that all states do not require a bonding agent. Do not let the bonding agent dry before the patch material is placed. (Follow the manufacturer instructions on bonding agent).
7. Use a trowel to firmly apply the patch material into the void created by the spall.
8. Use trowels to sculpt the member shape and texture the finish.
9. Spray apply a curing compound or wet cure for 7 days over the patched area, unless latex-modified concrete patch material is used.

The most permanent patch product for Portland cement concrete is Portland cement concrete. The closer the physical properties of the patch material are to the existing material, the better the patch will perform. Minimize the shrinkage of the patching material by limiting water to cement ratio when mixing. The inclusion of latex in a concrete or mortar will help reduce the amount of water required for workability and also reduce the permeability of the patch. Latex-modified or silica-fume concrete is recommended for vertical and inverted concrete patches due to their enhanced bond strength and cohesiveness. Other concrete additives can be used to reduce setting time and increase strength. For thin patches, the aggregate can be adjusted in the mix design.

11.4.2.2 Recasting with Concrete

Larger concrete superstructure areas can be effectively repaired by replacing the damaged or deteriorated area. Recasting concrete requires formwork and is common for larger beam repairs. Examples of this repair are shown in Figure 11.17 (during) and Figure 11.18 (after).

Figure 11.17 Forming Repair. Pumping Concrete into the Form (Courtesy of the Alberta Ministry of Transportation)

Figure 11.18 Completed Form Repair

Suggested Procedure

Recasting

1. Consult an engineer to determine if there are any structural capacity concerns from removing unsound concrete to the depths and limits necessary for the repair. Place any required temporary shoring or bracing necessary to support the structure during the repair.
2. Sawcut the perimeter edges straight to a depth of ¾ inch. Remove any loose concrete in the area to be patched. Concrete should be removed 1 inch all around exposed rebar whenever possible.
3. The existing surface should be cleaned by light sand-blasting. The concrete surface should be saturated with water spray, if dry, and then allowed to return to a surface dry condition. This will prevent the old concrete surface from absorbing the new concrete mix water. (Continued on next page).

Suggested Procedure

Recasting Procedure (Con't)

4. Install the concrete formwork. The formwork should be rigid enough to prevent new concrete from sagging away from the existing concrete under the weight of new concrete. The formwork should withstand forces from concrete pumping and the vibrating used to consolidate the concrete. Plywood is often used for concrete formwork. Steel forms can be used but they are heavy and not easily handled. Forms are typically attached to the member being repaired or hung from the structure and should be well constructed to prevent leakage of the patch concrete.
5. Prior to placing the concrete, the forms should be cleaned, sprayed with a form release agent and wetted to prevent absorption of the water used in the concrete.
6. Apply a bonding agent (usually a cement grout) onto the concrete surface just before the installation of formwork. It is very important that the bonding grout does not dry out before the repair concrete can be placed. A dry bonding grout can destroy bond of the new concrete to the existing concrete. For this reason many owners do not allow bonding agents. The use of specially formulated polymer bonding agents may be required if the formwork cannot be placed before the grout will dry.
7. Place the new concrete through holes in the top of the formwork for vertical patches if the top is not accessible. Inverted patches should be cast from above if possible through fill holes in the member. If inverted patches cannot be recast from above consider using the shotcrete repair method. Concrete for recasting should easily flow and fill all the voids in the form. Typically 3/8 inch course aggregates are used in the mix to improve flow and consolidation. Limit the water to cement ratio to avoid shrinkage cracking of the repair. Concrete additives may be used to provide workability without resorting to adding additional water.
8. Internally vibrate the newly placed concrete through the fill holes in the forms or by vibrating the forms from the outside. Vibration should be done along the length of the repair after shallow lifts of concrete have been placed. Good compaction is achieved by placing the concrete in small amounts and vibrating effectively as the work proceeds. An option to vibration is to use self-consolidating concrete which does not need any vibration.
9. Allow the concrete to cure.
10. Remove the form work and grind off any excess concrete or fill any voids that formed.

11.4.2.3 Prepacking Dry Aggregates and Grouting

This repair method is very similar to recasting. The surface preparation and formwork is basically the same as those for recasting concrete. The only difference is that a uniform size dry aggregate is packed in the space behind the form so that it fills the space completely. Grout is then pumped from the lowest to the highest point to fill the space between the aggregate, as shown in Figure 11.19. An advantage of prepacking dry aggregate and grouting is that the overall shrinkage of the repair is greatly reduced.

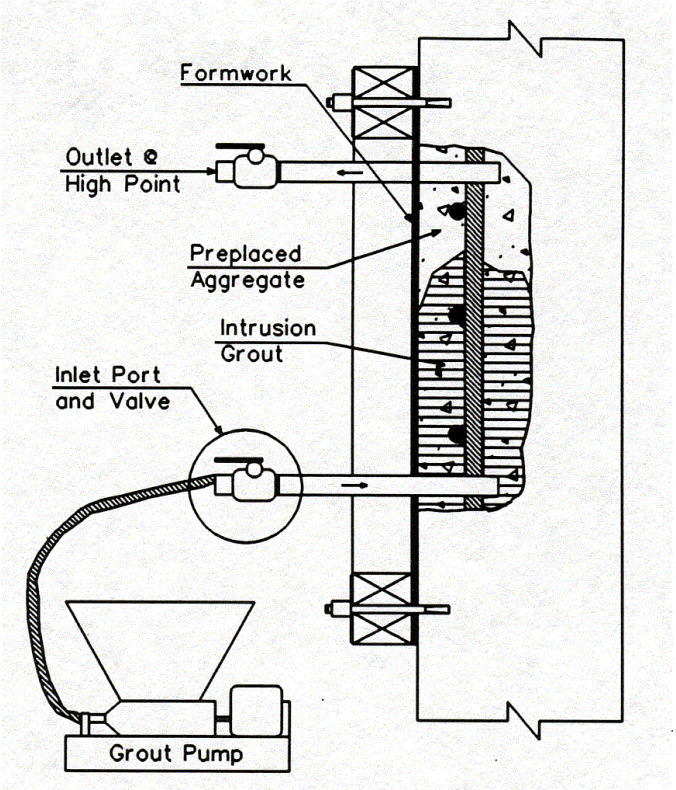


Figure 11.19 Repairing with Dry Aggregates and Grout

11.4.2.4 Shotcrete Repair Method

Shotcrete, or pneumatic applied mortar, is used to repair and restore the surface of concrete bridge elements. It is conveyed through a hose and nozzle and pneumatically projected at high velocity onto a surface. It contains the same cement, aggregate and water as concrete except that there are no course aggregates in the mix. Compaction is achieved by the velocity of the mixture when applied. Shotcrete repairs require a highly trained operator to obtain long lasting results. Many bridge owners require submission of operator qualifications and test panels prior to starting work.

The mix has high cement content and low water/cement ratio. The addition of silica fume, fly ash and/or slag can enhance the performance of shotcrete. Steel or synthetic fibers have also been used to increase tensile strength and decrease the potential for cracking. When properly applied, the mortar is dense, durable, and has superior bonding characteristics.

Shotcrete is desirable on vertical and overhead patches because no forming is required and the pneumatically applied mortar can repair large surface areas in relatively short periods of time. The shotcrete may need to be anchored to the existing reinforcement with additional rebar, wire mesh or mechanical anchors. Shotcrete should not be used for repairs of less than 1 inch thick. A photographic example of shotcrete repair is shown in Figure 11.20, and a procedure for shotcrete repair follows.

Figure 11.20 Shotcrete Application

Suggested Procedure

Shotcrete Repair

1. Preparation of the existing surface is an important part of the repair. The edges of the repair area should be sawcut at least ¾ inch deep at a 45 degree angle into the repair area to prevent rebound of the shotcrete material. All deteriorated concrete should be removed to a minimum of 1 inch behind exposed reinforcement. All surfaces should be cleaned with high pressure water or by sand blasting.
2. For repairs 3 inches or deeper, welded wire fabric or wire mesh should be mechanically affixed to the existing concrete surface prior to the placement of the shotcrete. The wire mesh will help insure the integrity of the repaired area and limit cracking.
3. The existing surface is wetted so it will not absorb water from the pneumatic mortar.
4. Apply the shotcrete. A thin bond coat should be applied first with subsequent layers building up the desired thickness. When applying, place the shotcrete nozzle at 90 degree angles to the repair surface whenever possible. Corners should be applied at a 45 degree angle to prevent rebound. Maintain a uniform flow of material and limit the layer thickness to prevent sags or sloughing to occur. The natural handgun finish is preferred from bond and durability standpoints. Scraping or cutting may be used to remove high points and material that has exceeded the limits of the repair after the mortar has become still enough to withstand the pull of the cutting device. Troweling or other surface finishing is discouraged as it has a tendency to disturb the bond.
5. Curing is very important for the rich mixes and thin sections used with pneumatic mortar. Seven days of water curing is generally advisable to promote good hydration of the cement, keep the mortar cool in hot weather, and prevent early shrinkage that may disturb the bond.

11.4.3 Epoxy Injection Crack Repair

Cracks that develop in concrete superstructure elements can be effectively repaired using epoxy injection techniques. Epoxy injection techniques are well suited for cracks caused by a variety of stresses, but not cracks caused by corrosion of reinforcing steel. Epoxy repairs are typically recommended for crack widths of 0.002 inches or wider. An example of a completed repair is shown in Figure 11.21. Detailed procedures for epoxy injection are provided in Chapter 5.

Figure 11.21 Completed Epoxy Crack Repair

11.4.4 Beam End Repair Procedure

Reinforced concrete beam ends can experience cracking and spalling near the bearings from leaking joints that cause corrosion, freeze thaw damage, or from thermal forces that develop when the bearings do not allow adequate movement. An example of beam end deterioration is shown in Figure 11.22.

Figure 11.22 Beam End Deterioration

A procedure for reinforced beam end repair follows below. It can also be applied to prestressed beams, as long as an engineer is consulted.

Suggested Procedure

Reinforced Concrete Beam End Repair

1. Restrict the vehicle loads on the affected beam by directing traffic to the far side of the bridge until repairs on the beam end are complete.
2. Determine if the existing substructure can be used to jack the bridge up or if a jacking bent will need to be constructed. The jacking supports and jacking procedure should be reviewed by an engineer before any lifting begins.
3. Place jacks and raise the entire end of the bridge a quarter of an inch (Figure 11.23). The lift should only be enough to take the load off and to allow a piece of sheet metal to be inserted on the beam seat as a bond breaker for the new concrete. Check with engineer if this step is necessary
4. Sawcut the concrete edges in a stepped fashion to avoid feathered edges and to provide bearing surfaces for the new concrete (see detail in Figure 11.24).
5. Remove the deteriorated concrete in steps.
6. Epoxy inject any visible cracks not within the removal limits following the epoxy injection procedure in this section.
7. Place new reinforcement as needed, making sure it is properly lapped, anchored, or mechanically attached to the existing steel. Bars can be welded to the existing longitudinal bars as shown in Figure 11.24.
8. Clean repair using abrasive blast cleaning.
9. Apply epoxy bonding agent to prepare the surfaces of the beam end. If agency does not allow a bonding agent, bring surface to saturated surface dry condition.
10. Place the forms for the new concrete. Place the new concrete. A non-shrink additive should be used in the new concrete.
11. After the concrete has reached sufficient strength, jack all beams simultaneously to sufficient height to allow the new bearings to be placed or to reinstall the old bearings.
12. Uniformly lower the end of the bridge. Check for possible distress in the repaired area.
13. Remove the jacking system.

Figure 11.23 Beam End Repair Preparation

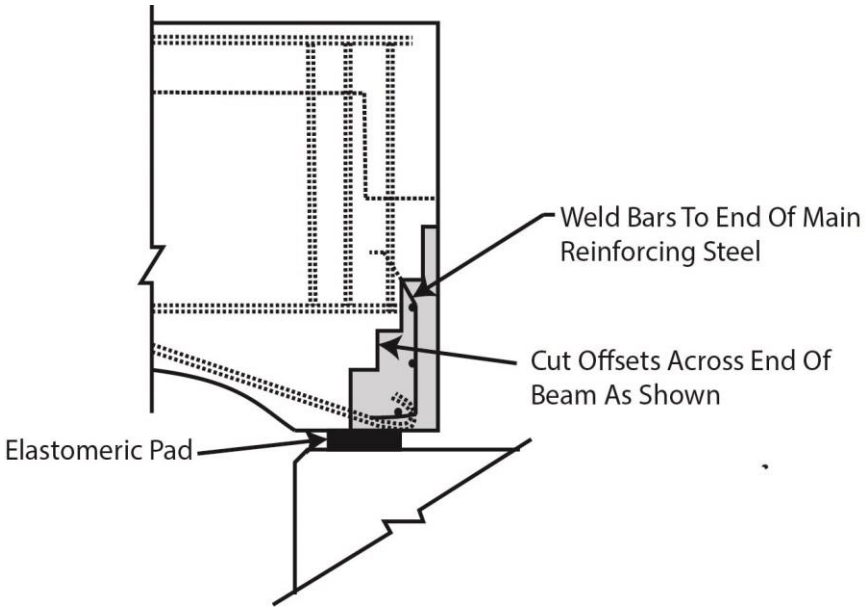


Figure 11.24 Detail of Repaired Beam End

11.4.5 Fiber Reinforced Polymer Repair

The flexural and shear strength of concrete superstructure members can be increased or restored by applying externally bonded fiber reinforced polymer (FRP) strips that supplement the tensile steel within the member. An example of this procedure is shown in Figure 11.25. This repair method can be highly effective in restoring strength in beams, stringers and floor beams that have lost strength due to cracking or loss of reinforcing steel caused by impact or deterioration. It has been successfully used as a preventive maintenance application for the beam ends of precast prestressed concrete girders.

The FRP strips are typically bonded to the bottom of beams in areas of high tension or the sides of beam webs in high shear areas. An engineer should determine the location, number and width of FRP strips to be placed. Once the number and placement of the FRP layers has been determined, the FRP can be applied. A suggested procedure for FRP application follows:

Suggested Procedure

FRP Application

1. Remove any unsound concrete that is within the area the FRP is to be applied.
2. Cracks wider than 0.010 inches wide should be filled with epoxy resin following the epoxy injection procedure in this section.
3. The concrete surface should be thoroughly cleaned using abrasive hand tools or blast equipment. The surface should be blown clean with compressed air.
4. The FRP should be cut to the size with heavy shears as specified by the engineer and in accordance with manufacturer's recommendations.
5. Prime the clean and dry concrete surface with epoxy resin using a trowel. The primed area should exceed the FRP size by approximately one half inch on all sides. Allow the epoxy to become tacky.
6. Prime the surface of the FRP to be placed on the concrete.
7. Place the FRP strips on the primed concrete such that the epoxy primed sides stick together epoxy to epoxy (Figure 11.25).
8. Use a rubber roller to press the FRP flat and smooth on the concrete.
9. Allow the epoxy resin to fully cure.

Figure 11.25 Applying FRP to Strengthen a Beam

11.4.6 Repairing Deteriorated or Damaged Reinforcing Steel

Concrete superstructure elements are typically reinforced with conventional reinforcement (rebar) or prestressing strands. The reinforcement can sustain section loss due to corrosion or can be damaged from vehicular impact. The repair techniques used to restore the reinforcing varies depending on whether the steel is stressed or not.

Conventional Rebar

When performing concrete repairs, it is important to protect the existing steel reinforcement. Incorporating the existing reinforcement into the new repair material will lengthen the service life of the repair. Steps to incorporating the existing reinforcement include:

- Locate existing reinforcing steel
- Remove unsound concrete from around reinforcing steel
- Assess damage to existing steel
- Clean
- Incorporate into repair

Locating the existing reinforcing steel is very important. If the concrete removal is started without knowing the depth of concrete cover over the steel, damage to the steel could result. There are non-destructive testing methods, such as using a pachometer, that determine cover depth.

Unsound concrete should be totally removed from the perimeter of the existing reinforcing. Concrete needs to properly bond to the reinforcing steel to resist forces. If more than one half of the diameter of the bar is exposed, the entire bar should be exposed including the underside. The underside removal could be determined based on the size of the aggregate of the repair concrete, but usually a 1 inch clear cover is needed. Concrete removal or existing corrosion may reduce the cross sectional area of the reinforcing bar to a point where it should be totally removed and replaced. That can be determined with the aid of an engineer.

Reinforcement bar cleaning is essential for concrete bond. This includes removal of rust and chlorides from the steel to inhibit future corrosion. Ways to accomplish this for bridge superstructures include wire brushing or sand blasting.

Once bars are cleaned they can be incorporated into the new repair. If supplemental reinforcement is needed to replace damaged or deteriorated bars, the new bars should be connected to existing reinforcement with proper lap lengths or mechanical connections. A lap length means the new reinforcement is placed side by side with the existing for a predetermined length decided by the bar size. Most agency specifications provide these lengths. If the patch is not large enough for a full lap length, mechanical bar splices can be used.

Prestressed Strand Splicing

Girders with significant deterioration or damage from truck impacts have been successfully repaired using the spliced strand technique. The area requiring a strand splice must be away from bearing areas and strand hold-down points. The strand to be spliced must be anchored beyond the repair area so that the re-stressing force can be attained. The length for anchorage needs to be confirmed by the engineer and is called the strand development length. The equipment for splicing strands will not easily accommodate abrupt changes in strand profile limiting the use of this technique to areas where the strands are relatively straight.

The strands in prestressed concrete members have been tensioned, effectively squeezing the concrete and placing it in compression. The tendons place sufficient compression in the concrete to prevent the section from going into tension when the bridge is loaded.

Cracks in prestressed concrete and corrosion of the strands are concerns. Since the tension is high in the tendons, the loss of a significant amount of concrete can cause the remainder of the

section to crush and fail. A single tendon break due to an impact or corrosion will typically only cause a minor effect, however several tendons snapping may cause a sudden failure of the member.

Special sequenced steps are needed when repairing prestressed concrete members to restore the tension that existed in the strand originally. These steps are normally determined by a structural engineer. Tension must be placed back into broken tendons as part of the repair using tendon splices. They are anchored to the ends of the damaged tendon and a threaded coupler nut between the two anchors is torqued a prescribed amount to produce the tension needed in the tendon. It is desirable to maintain the original girder cross-section to maintain headroom below and for girder appearance. The splice hardware has a much larger diameter than the strand, reducing concrete cover and increasing congestion in the repair area. It may be necessary to offset splices to reduce congestion and facilitate concrete placement. Splices generally reduce the fatigue life of the strand and the member. Often times only the slack needs to be removed from repaired broken strands, for example when only a few strands are broken due to an overweight load.

Details showing the removal and installation sequence are presented in Figure 11.26 through Figure 11.30. A prestressed strand splice procedure follows:

Suggested Procedure

Prestressed Strand Splice

1. Consult an engineer to determine if there are any structural capacity concerns from removing unsound concrete to the depths and limits necessary for the repair. Place any required temporary shoring or bracing necessary to support the structure during the repair.
2. Sawcut the perimeter edges straight to a depth of 3/4 inch. Remove any loose concrete in the area to be repaired. Concrete should be removed 1 inch all around exposed rebar whenever possible. The minimum length of concrete removal necessary in order to install all the strand splicing and tensioning devices is approximately six feet (see Figure 11.26).
3. Saw cut the broken strand to remove any frayed or damaged length. Leave at least 4 inches of strand exposed to install the splice devices.
4. Install splice hardware consisting of coupler, stressing gauge and tensioning device (see Figure 11.27 and Figure 11.28). The arrangement for these devices may be changed if it is more convenient.
5. Torque the splice hardware to tension the strand.
6. If a significant amount of concrete is replaced, the compression must be removed from the concrete in the beam around the damaged area. This is done by placing a calibrated load on the bridge while the new concrete is placed and is reaching its design strength. This step is known as preloading the member.
7. Preload member and replace the concrete using one of the techniques outlined in this section.

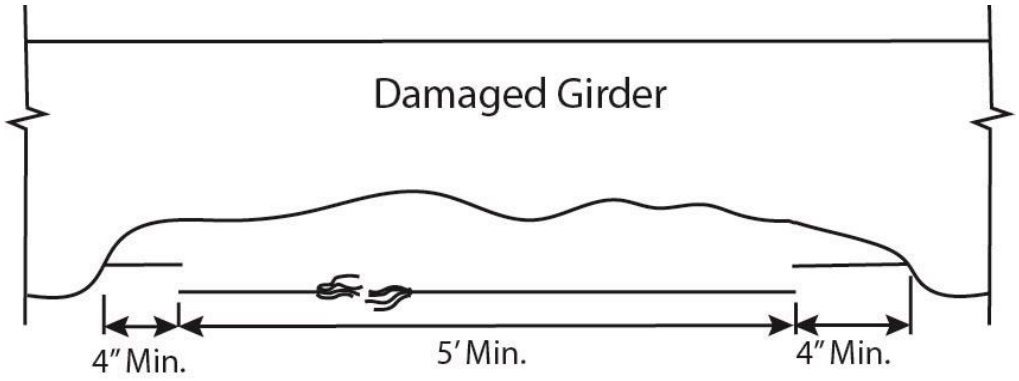


Figure 11.26 Remove Damaged Strand Length

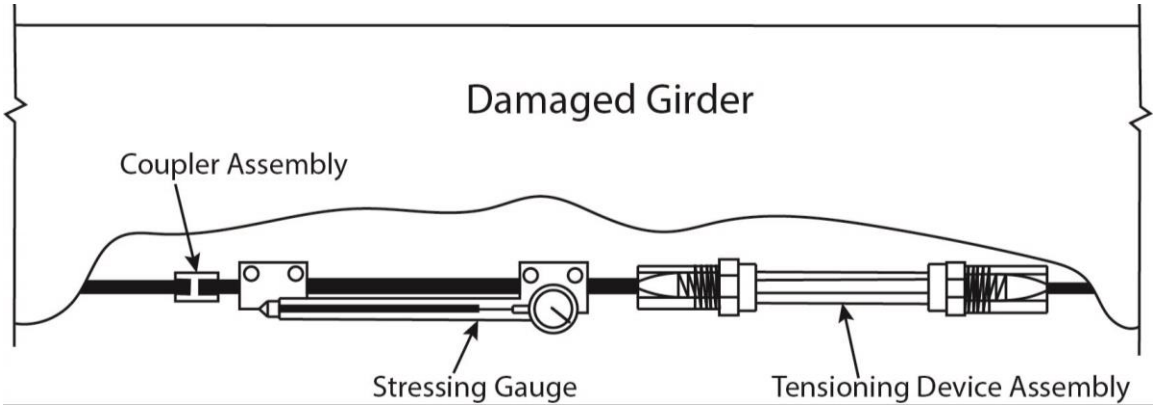


Figure 11.27 Splice Hardware

Figure 11.28 Tensioning the Splice (Courtesy of the Nebraska Department of Roads)

Figure 11.29 Completed Splice Ready for Concrete

An alternative to manually tensioning the splice is utilization of a coupling system. An example of a prestressed girder repair utilizing a coupling system is shown in Figure 11.30.

Figure 11.30 Completed Strand Coupling System in Prestressed Girder Repair

11.4.7 Maintenance and Repair of Adjacent Prestressed Box Beams

Prestress concrete adjacent box beams are a popular superstructure type. The beams are placed directly next to each other and may be connected with poured keyways and possibly transverse tie rods also referred to as post-tensioning. They may be constructed with or without a reinforced concrete deck cast-in-place over the beams. These beams can have issues with premature deterioration due to cracking and other maintenance issues depending on the construction details. Figure 11.31 shows the underside of an adjacent box beam bridge with significant water leakage between the joints which deteriorates the concrete. Maintenance recommendations have been provided in documents such as NCHRP Synthesis 393, *Adjacent Precast Box Beam Bridges: Connection Details*. According to research, the predominant distress observed in these bridges is reflective cracking of the slab along the shear keys between beams and the associated degradation below the cracks.

Figure 11.31 Underside of Adjacent Box Beam

Some potential preventive maintenance options for adjacent box beam bridges include

- Sealing the deck
- Removing the asphalt topping
- Sealing the cracks
- Washing the decks on an annual basis
- Unclog drains (see Figure 11.32)

Some potential repair options include

- Add a reinforced concrete deck
- Add supplemental transverse tie rods (post-tensioning)
- Replace the asphalt wearing surface with a concrete deck
- Use waterproofing membrane over the entire surface and reseal the deck

Note that these preventive maintenance or repair options are dependent upon whether the existing bridge had a poured concrete deck or not.

Figure 11.32 Clogged Drains are an Issue

11.5 Repair and Rehabilitation of Steel Superstructures

11.5.1 Introduction

Modern steel bridges typically require fewer steel superstructure repairs than older structures due to their shorter time in service; use of modern corrosion resistant steels with improved toughness; utilization of continuous superstructures with fewer deck joints or jointless bridges; absence of fatigue prone details and proper design to resist fatigue, "maintenance free" or low maintenance design and detailing of built-up members; adequate vertical and horizontal clearances reducing a potential for vehicle impact damage.

Hence, the majority of the steel superstructure repairs are performed on older bridges due to the following reasons:

- Many multi-span bridges have simple span superstructures with a large number of leaking deck joints
- Failure of the aging steel coating system
- Inadequate bridge drainage infrastructure – free drop spraying or clogged old downspout system
- Stress related fatigue in superstructures designed prior to 1974 and distortion induced fatigue in superstructures designed prior to early 1980's
- Many bridges are functionally obsolete due to low vertical under clearance and are susceptible to under passing vehicle impact damage, as well as narrow horizontal clearances on some through trusses, which can also lead to impact damage
- Corrosion occurs from roadway and high water debris collection on superstructure members, truss joints, beam seats, and around truss member penetrations through decks or sidewalks

Steel superstructure repair usually involves some type of welding and/or bolting. Both methods are discussed in Sections 13.5.1 and 13.5.2 and are also referred to many times within this chapter.

11.5.2 Corrosion and Section Loss

Section loss from corrosion is a common type of a steel superstructure damage that routinely requires maintenance repair. It often occurs at the girder ends under leaking deck joints and at the "pockets" of built up member connections, bottom flanges, and framing members that act as a shelf and trap debris.

11.5.2.1 Corrosion Damage at Girder Ends and Example Repair Procedure

Joint leaks over girder and stringer ends subject those areas to repeated moisture which degrades the steel coating and leads to corrosion and section loss. It typically affects the lower portion of girder ends, bearing stiffeners, and end diaphragms. See Figure 11.33 below for an example of girder end corrosion damage before (left) and after (right) sandblasting and primer application. After removing the corrosion product by sandblasting and priming, repairs can be performed.

Figure 11.33 Corrosion Damage at Girder Ends (Concrete Deck has been Removed)

When to Call the Engineer

Call the Engineer for plans and details regarding girder repairs.

A typical girder repair procedure is presented below.

Suggested Procedure

Girder Repair

1. Remove the paint within the limits of repair
2. Disconnect and remove existing end diaphragm(s) as required
3. Remove girder bottom flange to bearing welds by grinding
4. Jack the stringer 1/16 inch maximum and install temporary blocking
5. Remove deteriorated portion of the stringer to the required limits
6. Cut a tee or full section from a matching W section
7. Weld or bolt new tee section to replace the removed damaged portion of the stringer end or splice new full section using bolted splice connection. The choice of bolting or welding could be based on owner's requirements and environmental factors.
8. If bearing stiffeners were removed as part of the process, as shown in Figure 11.35 (left), the Engineer should be consulted to address how to replace them, if replacement is needed
9. Perform NDT (UT) testing and repair all weld flaws as required
10. Re-jack, remove blocking and lower the stringer onto the existing bearing
11. Field drill holes in the repaired stringer web and re-connect the existing or replacement end diaphragms using high strength bolts
12. Re-weld the girder bottom flange to the bearing sole plate
13. Clean and paint all steel surfaces disturbed by the repair

See Figure 11.34 and Figure 11.35 for representative examples of the above repairs.

Figure 11.34 Example of a Girder End Repair Utilizing Bolted Splice of a New Full Section

Figure 11.35 Repair in Progress using Tee – Section Cut Out From a Matching W Section

11.5.2.2 Repair of Built Up Members

Many older bridges have built up I-shaped steel girders, floorbeams, and other members that are typically comprised of plates and angles riveted together. An example drawing of a built up I-shaped steel girder is presented in Figure 11.36. This type of girder often traps debris between the rivets, stiffeners and on the top of beam flanges. The moisture and debris can come from leaking joints, inadequate scuppers, spray from vehicles, or bird droppings. The moisture and debris can lead to a failure of the protective coating, corrosion and ultimately section loss as shown in Figure 11.37.

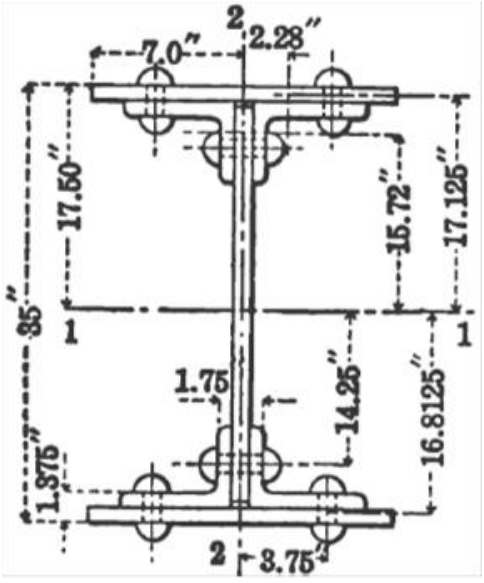


Figure 11.36 Typical Built-Up I Shape

Figure 11.37 Example of Heavy Corrosion and Section Loss

When to Call the Engineer

To assure the repair will restore the member to the desired capacity, an engineer should be consulted for design and detailing.

Highway overpass superstructures are particularly prone to moisture and debris accumulation carried by vehicle spray. While built up riveted members may be less forgiving, rolled members are also subject to corrosion from the same processes.

In cases where the section loss is minor and not significantly decreasing the capacity of the member, sand blasting and painting the area to arrest continued corrosion and section loss is often the chosen repair. In cases where section loss is significant and adversely affects the girder capacity, additional cover plates or angles may be added to restore the section as shown in Figure 11.38. To assure the repair will restore the member to the desired capacity, it should be designed and detailed by an Engineer.

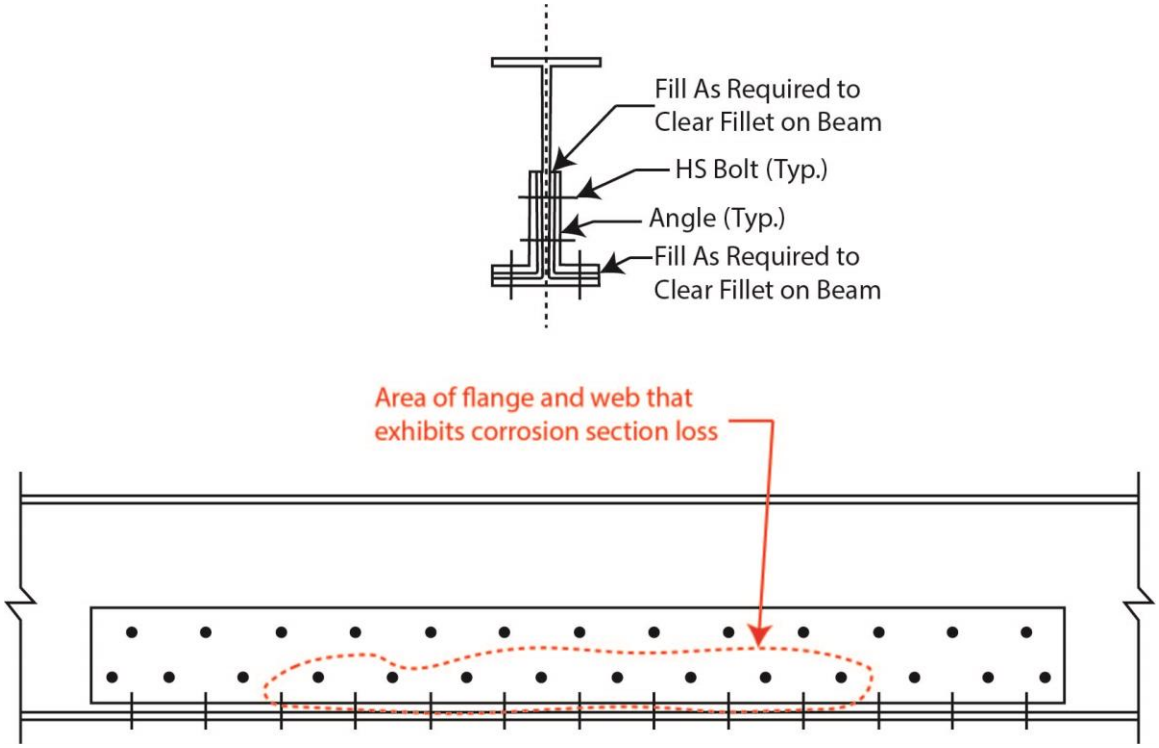

Fill As Required to Clear Fillet on Beam
HS Bolt (Typ.)
Angle (Typ.)
Fill As Required to Clear Fillet on Beam
Area of flange and web that exhibits corrosion section loss

Figure 11.38 Sample Girder Repair Detail

The photographs in Figure 11.39 show a girder repair made by the OBPA Maintenance Personnel (courtesy of the Ogdensburg Bridge and Port Authority (OBPA)).

Figure 11.39 Girder Repair. Before (left) and After (right).

Crevice Corrosion

Crevice corrosion between the cover plates and other elements of built-up members (particularly riveted members) is not uncommon. It often causes plate distortion and buckling, and sometimes even leads to rivet failure. Examples of crevice corrosion are shown in Figure 11.40, which are built up box members without top plate (left) and with top plate (right).

Figure 11.40 Built Up Box Member Corrosion

In most cases, plate replacement is required. Sometimes if the distortion is relatively minor, a power tool cleaning using a needle scaler and/or wire brush, followed by sealing and painting will slow the progression of corrosion. This type of repair is not permanent, so continued monitoring and maintenance will be necessary.

11.5.2.3 Repair of Truss Members

Primary Compression and Tension Truss Members

The repair of primary compression and tension truss members can be complex. It may require special access and substantial bracing. It should be properly designed by an engineer and performed by experienced agency staff, or a qualified contractor. See Figure 11.41 for replacement of truss bottom chord plate utilizing high strength thread bars and jacks for the load transfer and stability.

Figure 11.41 Repair of Bottom Chord Plate (left) Utilizing Thread Bars and Jacks (right)

Primary members smaller than those shown in Figure 11.41 may also be repaired with the addition of cover plates and angles, similar to the repairs shown in Figure 11.38.

When to Call the Engineer

Whenever work is being performed on truss members, an engineer should be called to determine the extent of temporary bracing needed.

Secondary Members

The repair and replacement of secondary members such as bracing is typically less complex than that of primary members, but still requires a detailed procedure and adequate bracing to provide for overall stability of the truss during repairs. See Figure 11.42 for an example of secondary member replacement.

Figure 11.42 Replacement of Bracing and Connection Plate. Before (left) and After (right)

Lacing Bars and Batten Plates

Lacing bars and batten plates were commonly used in older riveted structures for the webs of I-shaped built-up members or the sides of box sections. Batten plates can be used in combination with the lacing bars and may be located at ends of built up members. They can have a rectangular or U-shape to accommodate connections to other members.

Deteriorated or otherwise damaged lacing bars and batten plates can typically be safely removed and replaced in kind one at a time, following the procedure similar to the rivet replacement procedure provided in Section 11.5.2.4.

See Figure 11.43 for an example of a batten plate replacement detail.

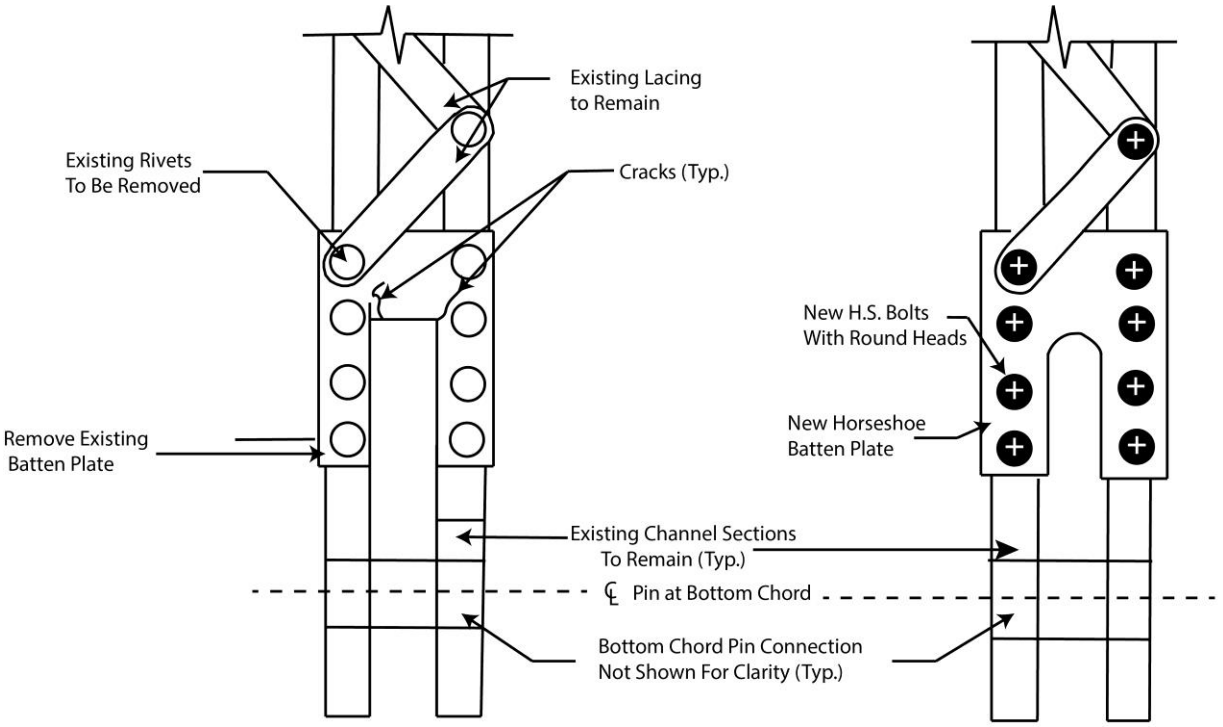


Figure 11.43 Batten Plate Replacement Detail

Repair of Truss Member Ends

Repair to the ends of steel truss members is a very common repair since these members are in close contact with salt and other contaminants on the bridge deck surface and because debris and moisture collect in congested end sections. This repair requires a sequence of events to properly remove, replace, and/or repair members.

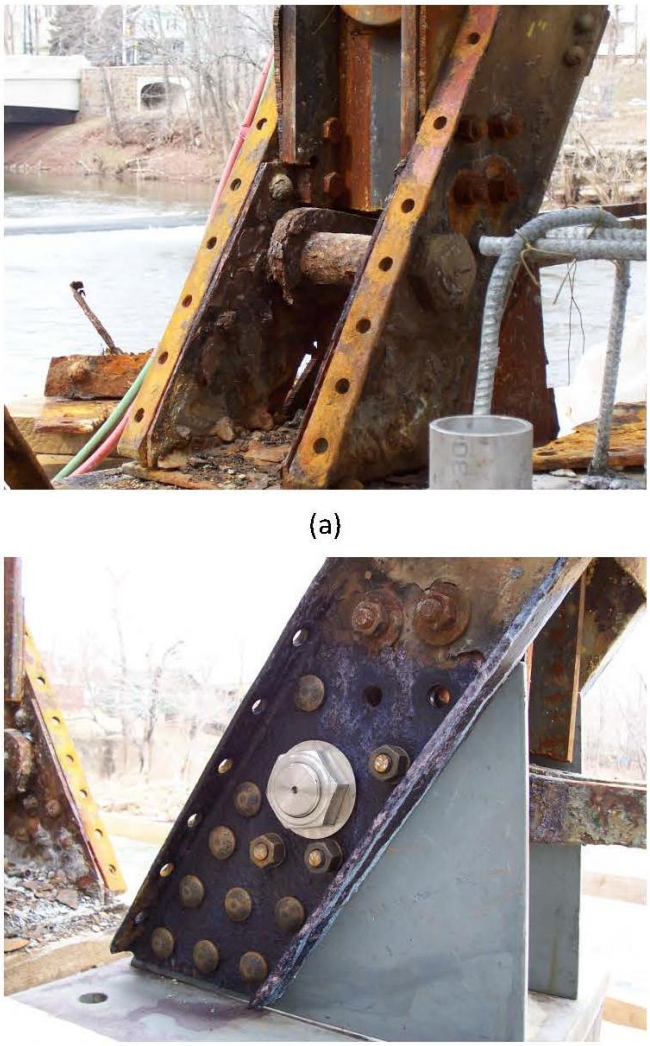
After temporarily supporting the connection, deteriorated members such as the pin, eyebars and gusset plates are removed, an example of which is shown in Figure 11.44. More specifically, Figure 11.44 shows (a) Pin and gusset plate replacement required due to severe corrosion and (b) New pin installed and gusset plate ready for installation.

(a)

(b)

Figure 11.44 Corroded Truss Member End Repair

In the replacement shown in Figure 11.45, pin and gusset plate are added. The eyebars were not replaced, but removed, repaired, and repainted.



(a)

(b)

Figure 11.45 Representative Corroded Truss Member End

In a third truss member end repair example shown in Figure 11.46, before and after photos of the eyebars and entire repair are shown. Specifically shown are (a) Corroded pin and eye bars, (b) Installation of new pin, eyebars cleaned and repainted (c) The repaired assembly is painted, (d) Representative location of repair (shown by arrow).

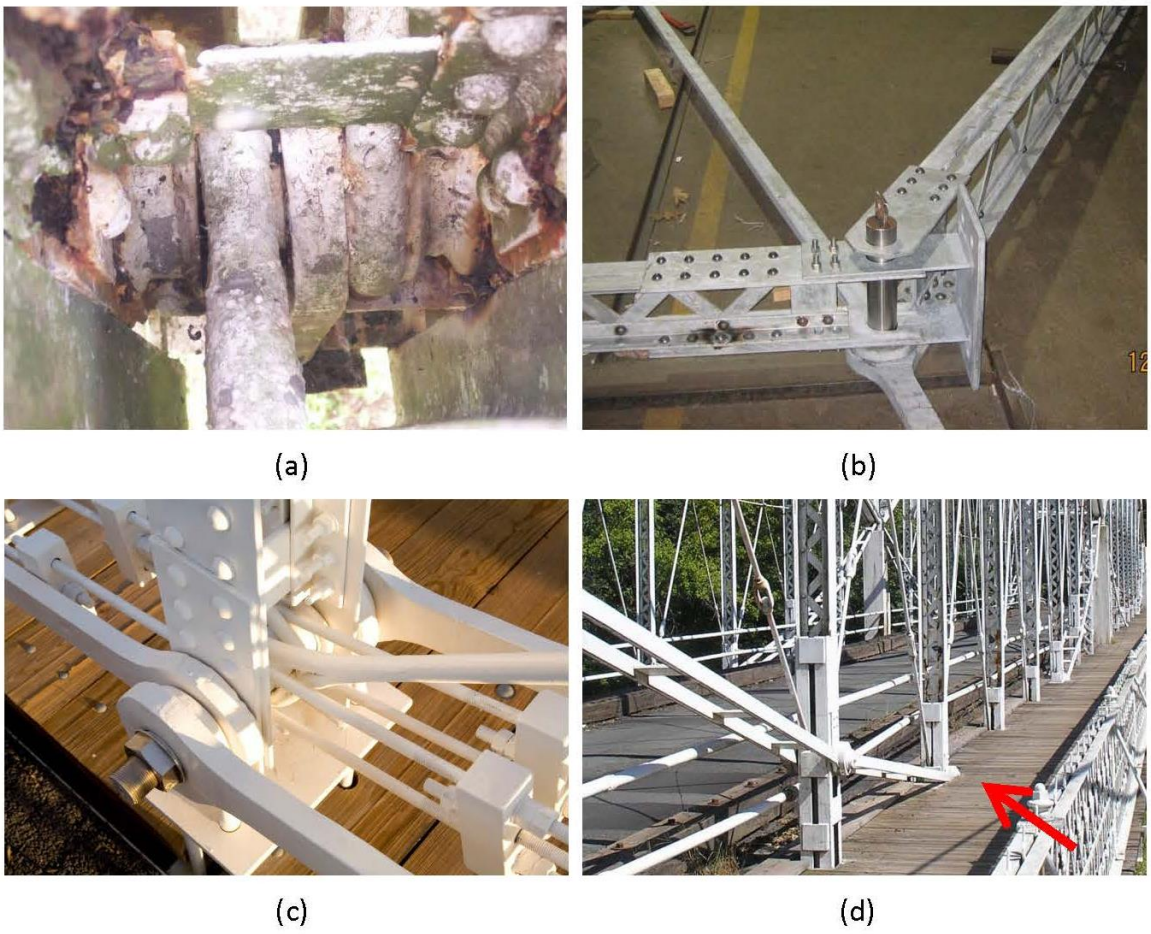


Figure 11.46 Truss Member End Repair

Repair of Primary Truss Members near Bearing

As stated earlier, the repair of primary compression and Tension truss members and their connections must be properly designed by an engineer and performed by experienced agency staff or a qualified contractor. Repairs will usually require substantial bracing and temporary supports if any parts of the primary members are being removed. These repairs can become more complex when they are in the vicinity of the bearings.

Figure 11.47 is a repair of the bottom chord near the bearing. This repair would require bracing, external support, and an engineered design. If repairs were being done on the pin or connection plate that attach the bottom chord to the diagonal member, that repair would require an even greater amount of bracing and shoring to remove the loads while the repair was being performed.

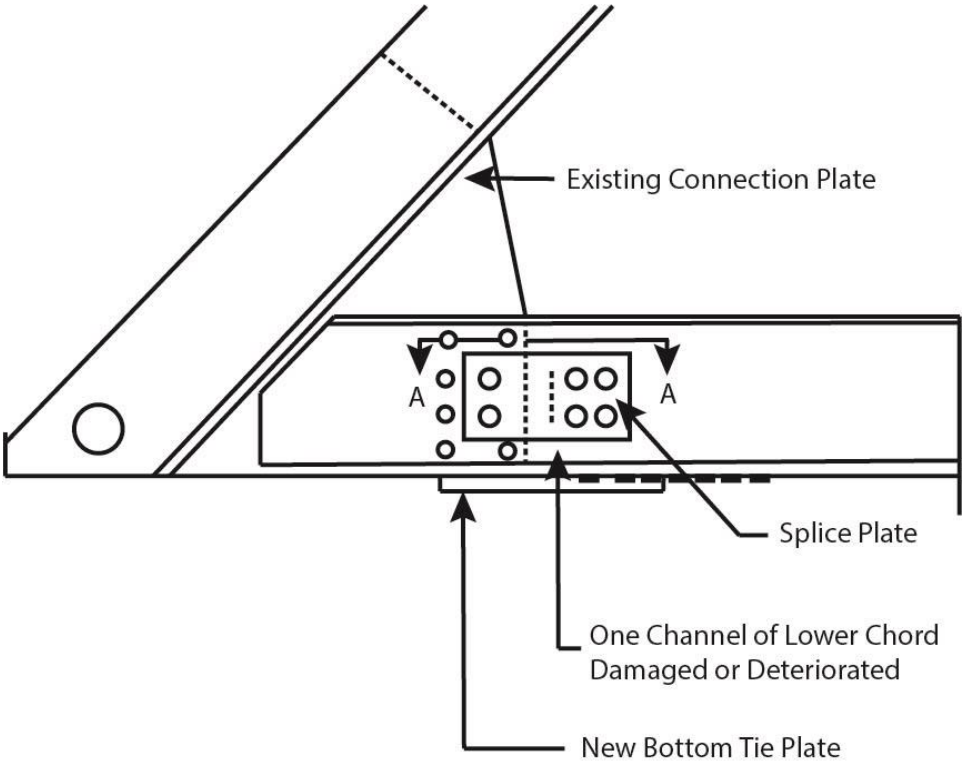


Figure 11.47 Schematic for Primary Chord Repair Near the Bearings

Bolted Splice Repair: Adding Doubler/Splice Plates

Another technique that can be used to repair corrosion loss and some types of cracks is doubler or splice plates, hereafter referred to as doubler plates or doublers. Doubler plates add material to either increase a cross-section or provide continuity at a deteriorated or cracked cross-section. For corrosion loss, the plates simply add enough steel cross section to make up the cross sectional loss. This technique can work as well for fatigue cracks to reduce stress range by allowing the plates to take the stress away from the cracked steel.

A consideration when using this repair for cracks is that the cracks will no longer be visible for inspection when plates are place on both sides of the repaired area. Although this repair is intended to prevent future crack growth, an engineer is needed to determine if crack propagation could be an issue and if this is the appropriate repair.

One problem with this type of repair can be maintaining the alignment of the two sides of the cracked section prior to the repair. The cracked surface may be buckled, making alignment difficult. Doubler plates should then be designed with the ability to straighten out any buckles in addition to providing added strength. The design process is identical to that used for a bolted field splice connection.

.

When to Call the Engineer

An engineer will be needed to design a bolted doubler/splice plate retrofit.

Doublers can be attached by welding or by using high-strength bolts. From a fatigue-resistance standpoint, bolted doublers are always better than welded, because a high-strength bolted connection can be considered an AASHTO Category B fatigue detail. A welded connection will most likely result in a Category E condition or worse. As a result, this manual recommends specifying only bolted doubler connections.

Suggested Procedure

Bolted Doubler/Splice Plate Retrofit

1. Align both sides of the crack if there is gross deformation
2. If there is a crack in the steel, drill out crack tip with a minimum 1.0 inch diameter hole to cease crack growth. A larger hole may be required to ensure you remove the crack tip, which is especially important because the repaired area cannot be subsequently inspected for crack growth.
3. Determine the required cross-sectional area of the doublers depending on whether the doublers will be intended to reduce stress ranges, restore a cracked section or corrosion loss. An engineer is needed.
4. Fabricate the doublers with all the holes drilled for bolted retrofits. In the field, clamp one of the doublers against the surface it will be attached to and use the doubler as a drilling template. Drill all holes in the cracked structure.
5. After all holes are drilled, remove the doublers and clean all contacting surfaces to remove oil, grease, dirt, and cutting fluids to restore the friction surface of the slip-critical connection.
6. Reposition the doublers and fill all the holes with high-strength structural bolts. Initially snug tighten all the bolts making sure all surfaces are mated. Use turn-of-nut method to fully tension all the bolts after the snug tight operation, or use tension-controlled bolts. See turn-of-nut requirements published by the Research Council on Structural Connections and Chapter 13 of this manual.
7. Prime and paint the bare surfaces. Document the location and repair procedure for inclusion in the bridge file for use in future inspections.

Figure 11.48 shows the completed Bolted Splice Repair.

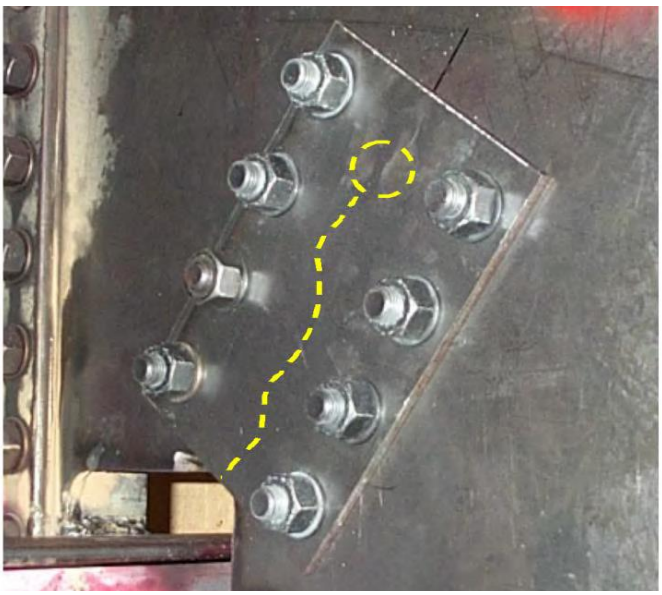


Figure 11.48 Completed Bolted Splice Repair

For more of these types of retrofits, please reference FHWA Publication FHWA-IF-13-020 *Manual for Repair and Retrofit of Fatigue Cracks to Steel Bridges*, March 2013.

11.5.2.4 Rivet Replacement

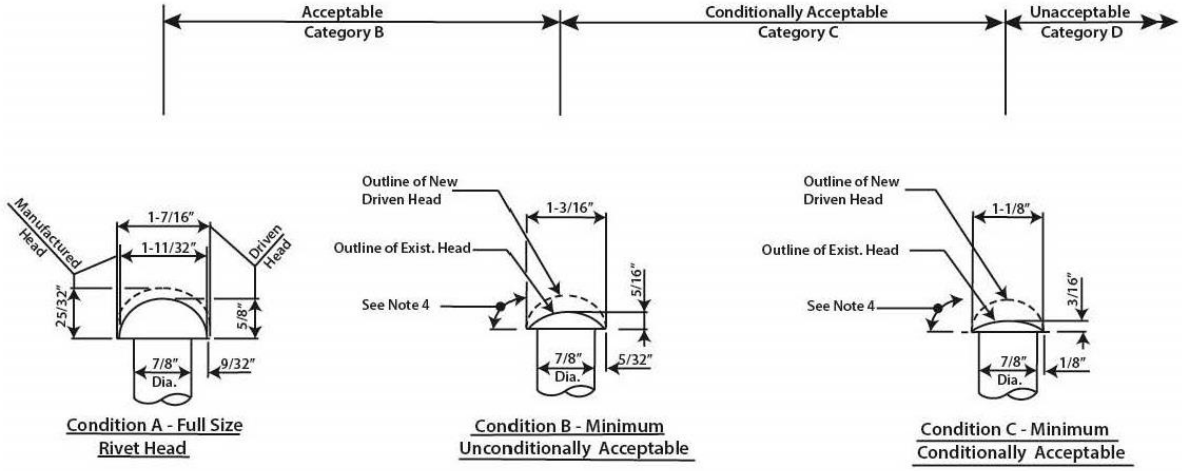
Built up members and connections often exhibit corrosion of rivet heads requiring their replacement. An example of corroded rivet heads is shown in Figure 11.49.

Figure 11.49 Rivet Heads Exhibiting Various Degrees of Corrosion

Rivet acceptance and replacement criteria varies between various agencies and Bridge owners. Rivets (even at the same connection) often have various degrees of head deterioration and volume loss. The question is: how much of the rivet head metal remaining is “enough” for the rivet to perform as intended and provide for the connection capacity before it shall be removed and replaced?

Figure 11.50 is a drawing of rivet acceptance criteria for 7/8 inch diameter rivets from the New York State Department of Transportation. These criteria were used on some projects and are

offered here as guidance. The figure outlines a new rivet head shape (dashed line) versus actual (remaining) rivet head that shall be field measured (solid line) and qualifies if the rivet should be considered for replacement. It divides rivet head conditions into 4 categories ranging from New to Unacceptable with corresponding qualifier notes that describe their acceptance criteria.



Rivet Replacement Criteria for 7/8" Dia. Rivets

Rivet Replacement Notes:

1. Rivets category B with dimensions of both heads meeting or surpassing each of the minimum requirements shown for condition B may be left in place subject to conditions described in notes 4 and 5.
2. Rivets Category C not meeting the requirements of condition B, but having dimensions which meet or surpass at both heads each of the minimum requirements shown for condition C may be left in place subject to the following conditions:
There is no prying action from applied stress or crevice corrosion which tends to separate the connected parts. Rivets' heads do not have additional losses described in Note 4.
Rivets may be left in place to the extent that their number does not exceed 20% of connection rivets in any one connection or 50% of stitch rivets in any one portion of a member.
Where the above percentages are exceeded the number of rivets over the prescribed percentage shall be replaced with high strength bolts.
When selecting rivets for replacement to meet the above percentage requirements, the worst rivets in any group or connection shall be selected for replacement.
3. Rivets category D not meeting the requirements of condition C a tether head shall be replaced with ASTM A325 high strength bolts of the same diameter, see Note 8 below.
4. Replacement will also be required for any rivet exhibiting additional loss in the form of pits or gouges at the edge of either head projecting beyond the shank where such loss reduces the section below the limits shown for Condition B.
5. Where crevice or interface corrosion between connected parts is present, the rivets adjacent to that area shall be replaced after cleaning between the parts regardless of the condition of the rivets.
6. Dimensions shown on these sketches for conditions B and C are minimum requirements for both driven and manufactured heads. The minimum height of head is measured to the center of the rivet. The minimum diameter applies to that direction in which it is the smallest.
7. All high strength bolt connections shall be assembled with a hardened washer under both the bolt head and nut. Where necessary washers may be clipped on one side to a point not closer than 7/8 inch of the bold diameter from the center of the washer.
8. Reading to dress up the rivet holes may be required; if after reaming the hole diameter exceeds tolerances the next larger sized bolt shall be installed.

Figure 11.50 Rivet Replacement Criteria for 7/8 inch Dia. Rivets and Notes

Rivets should be removed in a manner that will not damage the underlying connected material by using a pneumatic rivet breaker to shear the head. The shank may be driven out with a pneumatic punch or rivet breaker after shearing the head. If the rivet shanks cannot be removed by punching without damaging the base metal, the rivet shall be removed by drilling as shown in Figure 11.51.

Figure 11.51 Rivet Removal. Pneumatic Rivet Breaker (left) and Drill Removal (right)

One rivet at a time should be removed and replaced with High Strength (HS) Bolts. The replacement bolt should be properly tensioned prior to removal of a subsequent rivet.

When to Call the Engineer

With the consultation of an engineer, possibly more than one rivet can be replaced at a time.

Suggested Procedure

Replacement of Rivets with HS Bolts

Rivet Removal:

1. Break head of rivet with pneumatic rivet breaker, taking care not to damage the underlying metal.
2. Drive rivet shank out of hole using pneumatic punch. If punch will not remove rivet shank, drill rivet out of hole.
3. Remove only one rivet at a time and replace with HS bolt before removing other rivets unless specifically authorized by the Engineer.

Area Preparation:

1. Upon removal of each rivet, the base metal around the hole should be examined for surface irregularities, cracks, and deterioration.
2. All oxidized materials, nicks, and burrs, steel peaks and cusps that would interfere with the seating of the bolt head, nut and washers should be ground off.
3. Encrusted and built-up paint film around the head, nut and washers of new fastener should be removed to bare metal.
4. Where irregularities to the surface of the hole prevent normal alignment of the bolt or insertion of the bolt without damage to the threads, the hole should be reamed such that the holes are perpendicular to the surface.

HS Bolt Installation:

1. Replace rivets with bolts of the same nominal diameter.
2. Replacement bolts should be long enough to result in minimum stick-through of 1/16 inch beyond the nut.
3. Hardened washers should be used under both the head and nut.
4. If an irregular or beveled surface requires the use of a hardened beveled washer under a bolt head or nut, the beveled washer should be used under the element which is not turned. A hardened standard washer shall be used under the turned element.

11.5.3 Fatigue Crack Repairs

Fatigue damage is a structural damage to a steel member or its component resulting from cyclic loading leading to the initiation and propagation of cracks. Fatigue damage typically occurs at stress levels that are significantly lower than the base material yield stress.

Current design criteria and details have evolved to reduce the probability of both load-induced and distortion-induced fatigue damage. However, welded steel structures that were fabricated before early 1980's often used fatigue prone details which are more likely to exhibit load-induced and distortion-induced fatigue cracking that requires a remedy or repair.

An example of a fatigue crack is shown in Figure 11.52. Due to out of plane distortion where the floorbeam connects to the web, the crack most likely initiated at the lower floorbeam flange to web connection.

Figure 11.52 Fatigue Crack

The most common techniques that are used to remedy or repair fatigue cracking include weld grinding, arrest hole drilling, reinforcement plate strengthening and addition of a tee or double angle stiffener to flange connection. There are also other techniques as StopCrackEX bushing technology, peening, and release of connection. Weld Grinding, Arrest Hole Drilling, StopCrackEX bushing technology, and Reinforcement Plate Strengthening are discussed in this chapter.

For more information, please see NHI Course 134062 and FHWA *Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges*, found at:

http://www.fhwa.dot.gov/bridge/steel/pubs/hif13020/hif13020.pdf

11.5.3.1 Weld Grinding

Small cracks sometimes occur at the toe of a weld. These cracks mostly are due to weld shrinkage of larger size welds. Weld toe grinding and removal of the crack containing portions of these welds will increase the fatigue life of the weld and reduce the risk of crack propagation into the base metal. Weld grinding can be completed using disc or burr grinding depending on the access and preference. Grinding should be performed carefully to avoid creating a notch. The repair area should be tested using Magnetic-Particle Testing (MT) or Dye Penetrant after completion of the repair to verify the crack has been entirely removed.

Identification of which weld cracks can be ground verses repaired should be made by a trained person. Nondestructive testing can help to determine the depth of the crack. If superficial, grinding would be the preferred solution.

11.5.3.2 Drilling Arrest Hole

Drilling a crack arrest hole at the tip of a crack is a common repair method that has been widely utilized as an economical technique for arresting fatigue cracks. An example of the holes drilled by this procedure is shown in Figure 11.53. The size of the drill hole can be calculated by an engineer taking into account the calculated stress levels, thickness of steel and yield strength. However, this is rarely done. Holes are usually drilled at diameter of ¾ inch to 1 inch.

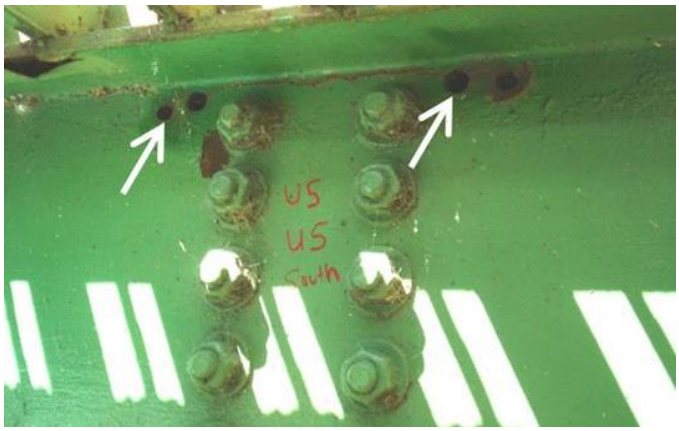


Figure 11.53 Examples of Drilled Crack Arrest Holes

The procedure must be done properly to be effective. The arrest hole should be drilled in front of the crack with the crack tip located between the hole edge and the center of the hole, as shown in Figure 11.54. Nondestructive testing such as dye penetrant testing or magnetic particle should be used to accurately locate the tip of the crack prior to determining the arrest hole location. The crack rarely propagates evenly thru the material thickness and often extends farther on one side of the plate than another. Therefore, dye penetrant testing on both sides of the cracked web allows a more accurate location of the crack tip. However, it may not always be practical or possible to inspect both sides due to limited access or physical constraints. After the hole is drilled additional dye penetrant testing should be used to test the hole perimeter for any cracks.

After the hole has been drilled, it could be left as-is or a bolt and nut can be inserted to plug the hole and help put the area around the hole in compression. Use of weld material to plug the hole, also referred to as a plug weld, is discouraged. These types of welds can cause additional cracks to develop in the base material, especially on vibrating steel bridge superstructures.

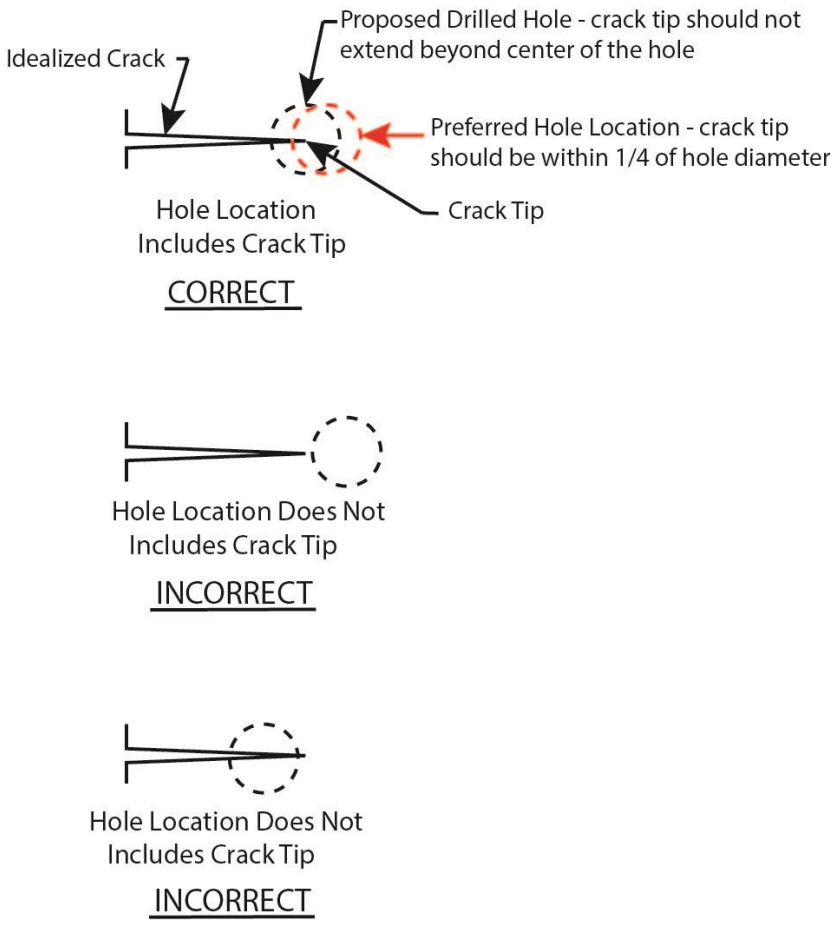

Idealized Crack
Proposed Drilled Hole - crack tip should not extend beyond center of the hole
Preferred Hole Location - crack tip should be within 1/4 of hole diameter
Hole Location Includes Crack Tip
Crack Tip
CORRECT
Hole Location Does Not Includes Crack Tip
INCORRECT
Hole Location Does Not Includes Crack Tip
INCORRECT

Figure 11.54 Drilled Arrest Hole Locations

11.5.3.3 StopCrackEX Technology

StopCrackEX Technology is used in military and aircraft industry to arrest fatigue cracks and more recently has been applied to bridge and building crack repairs. A cold-expanding bushing in the crack arrest hole creates a compressive field around the arrest hole. The additional compressive field created by the expanding bushing in the arrest hole further reduces the concentrated tensile stress field at the crack tip, mitigating crack growth. The StopCrackEX process is shown in Figure 11.55, where the gun is shown (left) and the resultant bushing and hole ahead of the crack tip is shown (right).

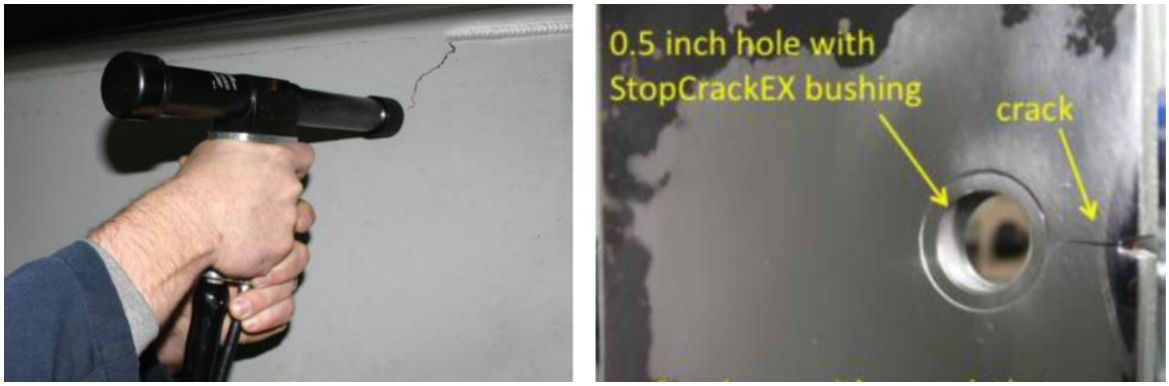


Figure 11.55 StopCrackEX Procedure

The StopCrackEx procedure utilizes the same approach as drilling the arrest holes, but requires smaller hole diameter (typically ½ inch) and is more effective due to the added benefit of localized pre-compression around the hole area.

11.5.3.4 Reinforcement Plate Strengthening

This repair provides strengthening by increasing the stress resisting area and reducing fatigue stress range stress. Strengthening or doubler plates can be installed on one side or both sides of the cracked member. Crack arrest hole should still be drilled at the toe of the crack and NDT testing is recommended prior to installation of the strengthening plate.

An example of an application for reinforcement plate repair is at a crack where a girder web has been coped. The photograph below, taken from the FHWA SHRP 2 Renewal Project R19A entitled "Design Guide for Bridges for Service Life", is a good example of such repair.

Figure 11.56 Example of Reinforcement Plate Repair

11.5.4 Impact Damage

Physical impact damage due to over height or errant vehicles is common. Figure 11.57 shows an example of bridge fascia girder damage that was struck by a vehicle passing under the bridge. The bottom flange and web were bent out of plane from the impact. Additional examples of impact damage are shown in Figure 11.58 and Figure 11.59.

Figure 11.57 Bridge Fascia Girder Damage Caused by Under Passing Vehicle (Courtesy of the New York State Thruway Authority)

Figure 11.58 Impact Damage to Fascia and First Interior Girder (Courtesy of VDOT)

Figure 11.59 Impact Damage that can be Repaired with Heat Straightening (Courtesy of VDOT)

The damaged element should be inspected for fractures. The initial inspection can be performed visually and it should be followed by NDT testing, such as Magnetic-Particle Testing (MT) or Dye Penetrant Testing, to verify the extent of the damage. If fracture damage is present, the member should be repaired using splice plates, another engineered solution, or the steel member should be replaced in kind. If there is no fracture damage, heat or mechanical straightening repair can be used effectively. These techniques are discussed in 11.5.4.1 Heat Straightening and 11.5.4.2 Mechanical Straightening, respectively.

Small gouges can be removed by grinding with radius and flared back at 1:10 slope using the procedure provided in *The Bridge Maintenance Manual*, PennDOT Publication No. 55, 2002.

Suggested Procedure

Small Gouge Removal by Grinding (PennDOT manual)

1. Grind impact area to bright metal to remove any regularities and surface defects. Using a sanding disc, smooth area and round over edges. Finish grinding should be done parallel to the stress to ensure that transverse grind marks are not present.
2. If the impact is within the proximity of a welded detail, the weld toes should be smoothed with a die grinder to ensure that no microcracks were introduced during the impact. Using a sanding disc smooth the area and round over edges. Finish grinding should be done parallel to the stress to ensure that transverse grind marks are not present.
3. The impact should be flared back to the material edge at not less than 1 to 10 slope.
4. The area should be thoroughly inspected including any weld toes of details within the vicinity of the impact using the ultrasonic or magnetic particle testing appropriate for the detail.

11.5.4.1 Heat Straightening

Heat straightening is a well-known technique used to repair a deformed steel member. The method utilizes heating and cooling with a specific heating pattern and heating torch type.

Areas of steel superstructure members that have been damaged can be repaired by heat straightening alone or in combination with applying external force. Repairs designated for heat straightening should be properly engineered and detailed. In addition, a detailed step-by step repair procedure that includes heating pattern, acceptable cooling method, maximum heating temperature, temperature control, limits of heating length/areas, need for and magnitude of any external force, QC/QA procedures and other governing parameters should be developed.

Properly executed and successful heat strengthening can restore a damaged superstructure element to its original condition, geometry and capacity. The FHWA *Guide for Heat-Straightening of Damaged Steel Bridge Members* is a good reference for heat straightening repairs. An example of a heat straightening procedure is shown in Figure 11.60.

Figure 11.60 Example of Heat Straightening Procedure

When to Call the Engineer

Heat straightening should only be performed by experienced personnel under direction of an engineer with experience. Also temperature should be closely monitored to ensure steel is not overheated.

The photograph in Figure 11.61 shows a nearly completed girder repair that was made utilizing heat straightening. The heating pattern is clearly visible in the photo.

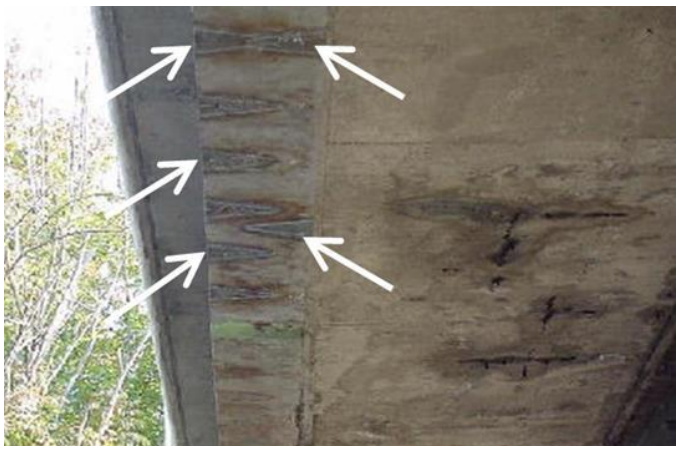


Figure 11.61 Nearly Completed Girder Repair Made Utilizing Heat Straightening (Courtesy of the New York State Thruway Authority)

11.5.4.2 Mechanical Straightening

Mechanical straightening, or cold bending, forces the deformed element into the original position by means of an external force. This is not a preferred method of repair, and it should be used on a limited basis for repair of minor and (most importantly) very localized damage. This type of repair has an increased risk of fracturing the damaged member when compared to heat straightening. Cold yielding also adversely affects mechanical properties of the base material. NDT testing (Magnetic Particle Test or Dye Penetrant Testing) of the repair should be done before and after mechanical straightening to verify that no cracks were created by the damage or the repair.

11.5.5 Tack Weld Removal

Tack or erection welds are widely used to temporarily hold connecting elements in place during construction. If not removed, they may develop fatigue cracks that could propagate into the base metal. Tack welds can be simply be removed by grinding followed by NDT testing. On completion of grinding, the area should be smooth, free of notches and any other surface defects that would potentially lead to development of a fatigue crack. The removal should not extend more than 1/16 inches into the base material without approval of the Engineer.

An example of a cracked tack weld that requires removal is shown in Figure 11.62.

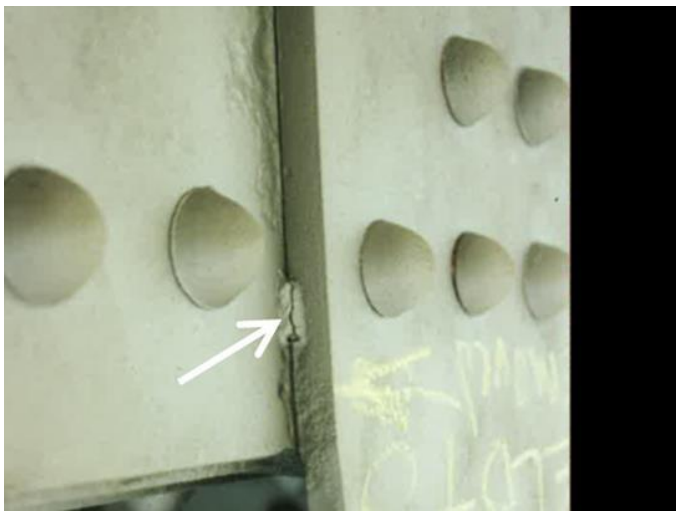


Figure 11.62 Cracked Tack Weld Requires Removal

11.6 Repair and Rehabilitation of Timber Superstructures

Repair and rehabilitation of timber structures involves one or more of the following activities; clamping and splicing new timbers to the existing structure, member repair or replacement, and/or employing external post-tensioning.

11.6.1 Clamping

Clamping involves the installation of steel plates above and below the timbers, which are drawn together by nuts and bolts or threaded rods depending on the depth of the timbers being clamped. The goal is strictly to prevent further splitting of the timbers. Clamping will not reverse the split by forcing the split back to its original pre-split position. A schematic of clamping was previously shown Figure 11.12 (top). Stitching, shown Figure 11.12 (bottom), involves the use of bolts or lag screws to contain the crack and prevent the timber from splitting further. The difference between clamping and stitching is that stitching can be done without interference with the surrounding structure; however clamping is a stronger mechanical bond and has shown better performance.

11.6.2 Splicing and Scabbing

Splicing and scabbing involves the addition of new material, usually either timber or steel, to allow a transfer of load through connections and around a cracked or split timber. Figure 11.63 (top) shows a typical spliced repair, in which two new timbers are installed on either side of the existing timber, and are sandwiched by either bolts or threaded rods with nuts. Typically, the spliced member should be cut all the way through to permit the splicing plates to properly assume the intended dead and live load carrying. This also ensures that each splice plate is equally handling its share of the load.

Scabbing, shown in Figure 11.63 bottom, is similar to splicing except that it often runs most of the length of the existing timber and is often only on one face of the existing timber. Scabbing is done more often since it allows a timber beam to carry a higher live load. Figure 11.64 shows a photo of a timber member repaired by scabbing.

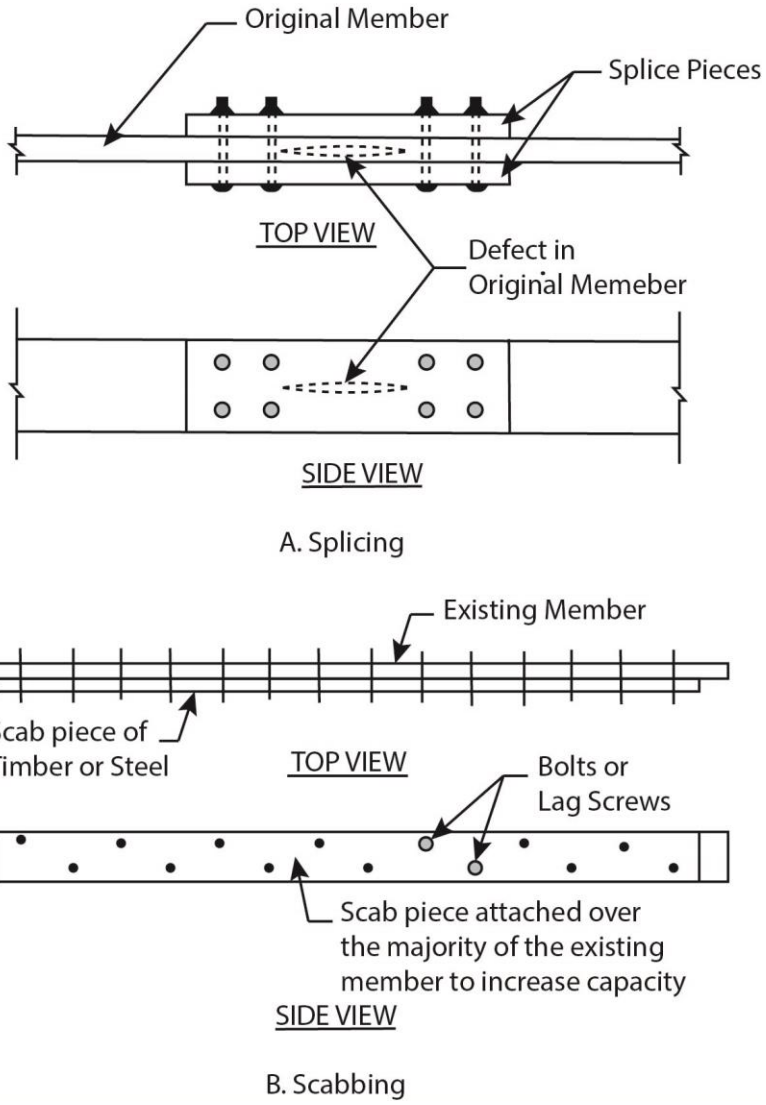


Figure 11.63 Splicing (top) and Scabbing (bottom)

Figure 11.64 Example of Scabbing an Existing Timber Member

11.6.3 External Post-Tensioning

Post-tensioning rods and cables have been used to strengthen or support existing timber members and to reduce or replace timber as truss members. These need to be checked during regular inspections.

Suggested Procedure

Post-Tensioned Rod and Cable Check

1. Note areas of corrosion and its severity. Significant section loss can lead to overstress due to higher tensile stress.
2. Check for broken members.
3. Pull or shake tensioning for looseness. If loose, cable tension has been lost.
4. Lightly tap rods and cables with a rubber mallet. Tensioned members will have a distinctive ring, non-tensioned members will not. Take care not to be struck by the rod or cable should it fail during the test.
5. Check anchorages for distress (cracks, strain, crushed wood).

In some cases, external post-tensioning is used to draw decks together similar to stress-laminating, but without the top surface being affected.

11.6.4 Repair of Timber Beams

Normally replacement is the most cost effective repair for decayed or damaged timber beams. A temporary repair can be made in some cases by turning beams with small damaged areas upside down, such as around the planking spikes. This may position the damaged area in a less stressed state, for example placing a damaged section in compression rather than tension. Often the repair will take as much time and effort as putting in a new beam and will result in a weakened beam with a short service life.

It is easier to replace a damaged beam if the deck is removed, but may not be practical if the deck is heavily spiked to all the beams, and only one beam is damaged. The usual practice is to install a new beam alongside the old beam, or one on each side. The new beam can be installed by cutting off one end of the beam at an angle so that it can be inserted into the space between the cap and the deck while clearing the opposite cap. After positioning the caps, the beam should be jacked up tightly against the deck, and hardwood or metal shims installed between the beams and the cap. The cut-off wedges from the angle cuts can be used as shims. The deck should then be spiked to the new beam. Figure 11.65 illustrates the replacement of timber beams.

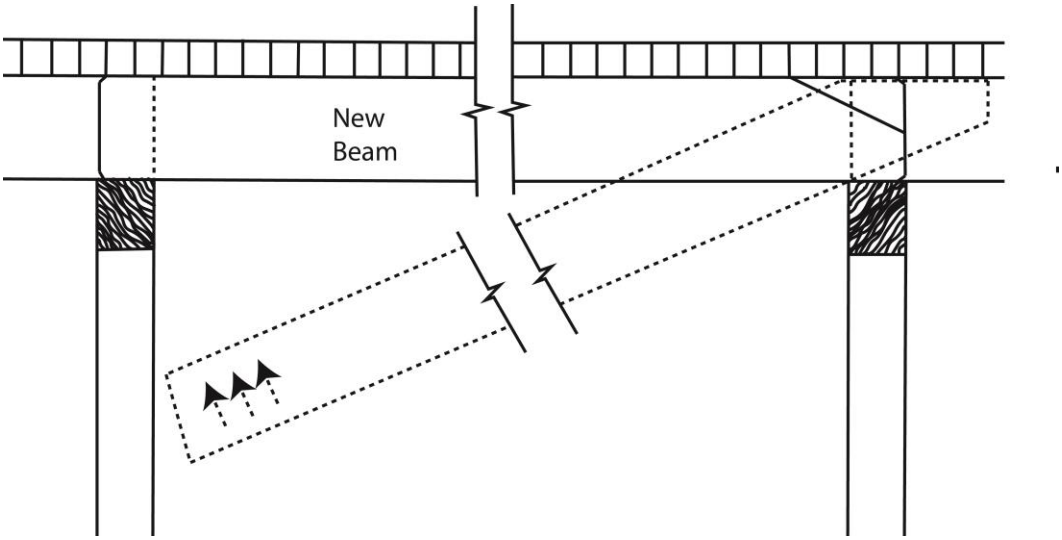


Figure 11.65 Timber Beam Replacement

Because of the difficulty in fitting a timber member of the same dimension into the available space, an alternative is to use a steel section that is smaller than the existing timber member, but has the same capacity or greater. Steel sections fit easily into the space and can be jacked into place. The sections can be rectangular shaped, round or any other shape that fits.

Suggested Procedure

Replacement of Timber Beam with Steel Support

1. A steel beam is installed between the bottom of the rectangular steel section and the top of the cap.
2. If needed, shims are tapped in between the top of the rectangular section and the deck for bearing.
3. Shims are tack welded into place.
4. For longevity, all steel members are hot dip galvanized ahead of time.
5. Welds or steel areas damaged during construction are sprayed with cold galvanizing paint.

For a photographic example of this type of replacement, please see Figure 11.66.

Figure 11.66 Rectangular Steel Section Replacement for Rotten Timber Girder (Courtesy of Washington DOT)

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Chapter 12 - Special Superstructure Elements

12.1 Common Types of Bridge Bearings

12.1.1 Foreword

Bridge bearings transfer loads from the superstructure (dead load, live load, and wind load) to substructure and allow beam end translations and rotations. There are two major categories of bridge bearings, those that allow for longitudinal translation (expansion bearings) and those that do not allow longitudinal translation (fixed bearings). Longitudinal translation is developed from changes in temperature, creep and shrinkage. This chapter will mainly discuss the maintenance and repair of expansion bearings, however, discussion will also be provided for fixed bearings where applicable.

What To Look For

- Debris
- Loose anchor bolts
- Missing/broken anchor bolts
- Bearing has moved on seat
- Corrosion
- Frozen bearings
- Inadequate lubrication

12.1.1 Mechanical Bearings

12.1.1.1 Sliding Bearings

Sliding bearings (Figure 12.1) allow beams to slide during expansion and contraction of the superstructure. Sliding bearings generally consist of a sole plate, sliding or rotation plate, and a masonry bearing plate that slide on each other. The flat surface allows for translation from expansion and contraction. The curved surface allows for beam rotation. The plates can be of similar or dissimilar materials. Sliding bearings can also employ the use of a dividing plate made of stainless steel, polytetrafluoroethylene (PTFE), or bronze to reduce friction between the surfaces.

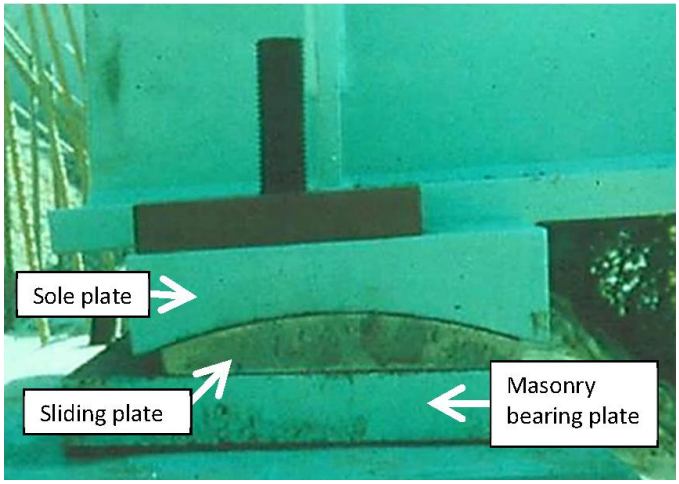

Figure 12.1 Example of a Sliding Bearing

12.1.1.2 Rocker Bearings

Rocker bearings allow expansion, contraction, and rotation of the superstructure by simply rocking back and forth. This type of bearing generally consists of a sole plate, pin, rocker and masonry plate. Rocker bearings can either be unpinned (Figure 12.2 Left) or pinned (Figure 12.2 Right).

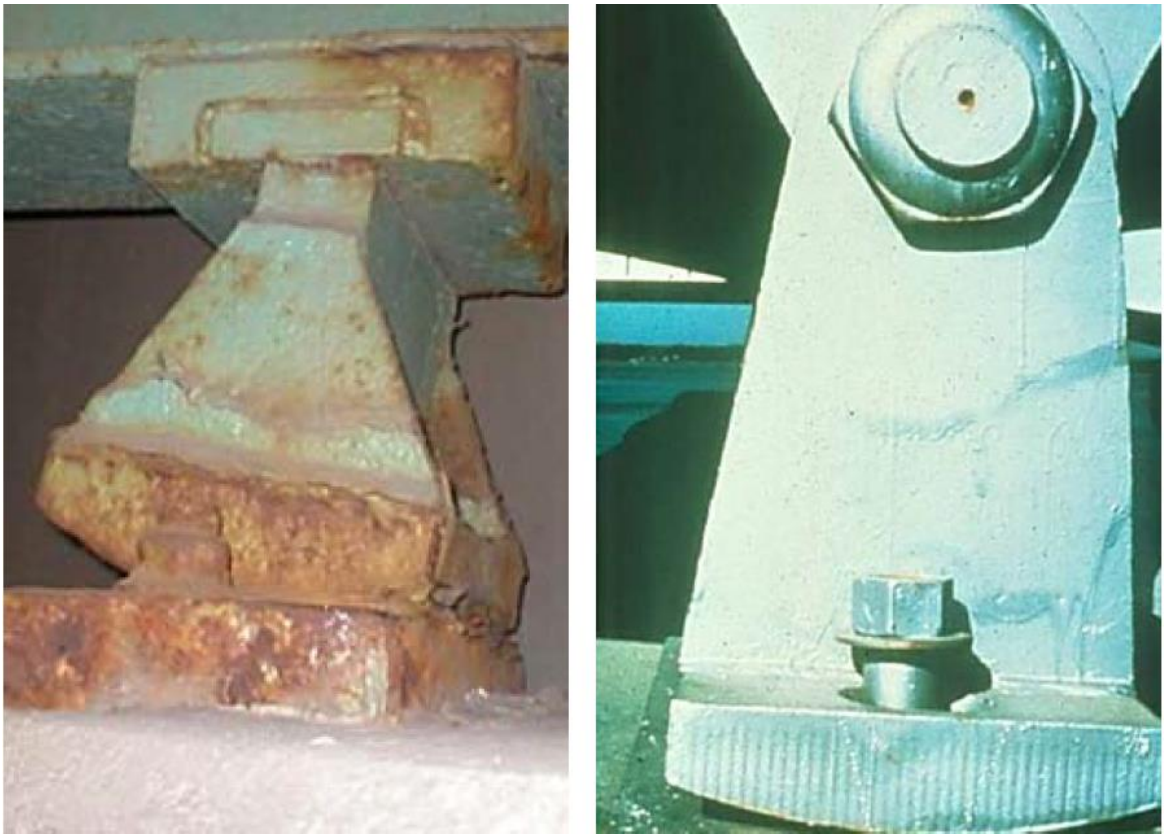

Figure 12.2 Examples of Rocker Bearings. Unpinned (left) and pinned (right)

A fixed pintle keeps the rocker in position on the masonry plate. The pintle is a vertical pin which is driven into the masonry plate at the shop that prevents the rocker from sliding instead of rocking. An example of a pintle is shown in Figure 12.3.

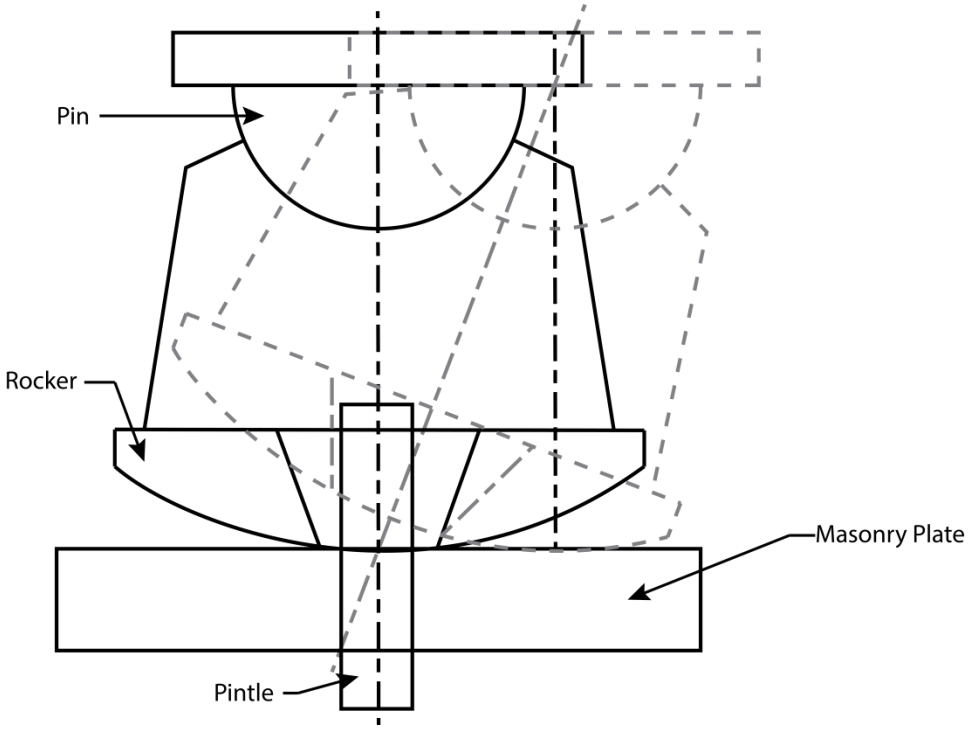

Figure 12.3 Schematic of Rocker Bearing Geometry Showing Pintle

12.1.1.3 Roller Bearings

Roller bearings generally consist of a cylindrical which bear and roll on a masonry plate. An example of a roller bearing is shown in Figure 12.4 (left). Other types of roller bearings will consist of a beam end which sits on a nest of encased ball bearings or rollers, an example of which is shown in Figure 12.4 (right). Generally, the encased roller bearing is difficult to maintain since access to the bearing assemblies is typically not achievable.

Figure 12.4 Roller bearings. Single Roller (left) and Nested Roller (right).

12.1.1.4 Pin and Hanger Bearings

Pin and hanger bearings are used for bridge trusses, girders and beams which connect a cantilevered span and a suspended span of a bridge where there is no substructure. An

example of a pin and hanger bearing is shown in Figure 12.5. Pin and hanger bearings are typically located under deck expansion joints where high deterioration can take place. This could lead to freezing pins and overstressing the pins, hangers, and chord members.

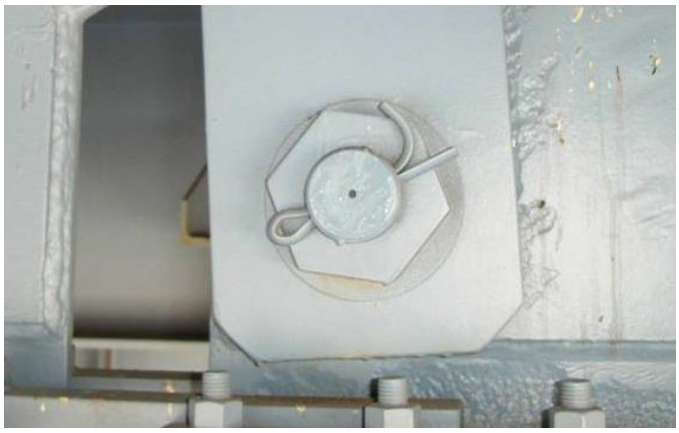

Figure 12.5 Example of a Pin and Hanger Bearing

12.1.2 Elastomeric Bearings

12.1.2.1 Plain Elastomeric

Plain elastomeric bearings consist of layers of elastic materials, typically rubber, which is compressed together. Elastomeric bearings can either be rectangular or circular. They may be flat or have internal beveled steel plates to accommodate steep profiles or beam cambers.

12.1.2.2 Reinforced Elastomeric

Like plain elastomeric bearings, reinforced elastomeric bearings consist of compressed layers of rubber, but will also contain layers of steel plates or fabric to reinforce them internally. An example of a reinforced elastomeric bearing is shown in Figure 12.6. The steel plates reinforcing the bearing can vary in thickness, but generally are around 11 gage thickness (0.1196 inch). Reinforced elastomeric bearings will generally be able to withstand higher loads and superstructure forces (e.g., dead, live, seismic, temperature, creep, shrinkage) than an unreinforced bearing. A schematic of a reinforced elastomeric bearing is shown in Figure 12.7.

Figure 12.6 Photo of a Reinforced Elastomeric Bearing

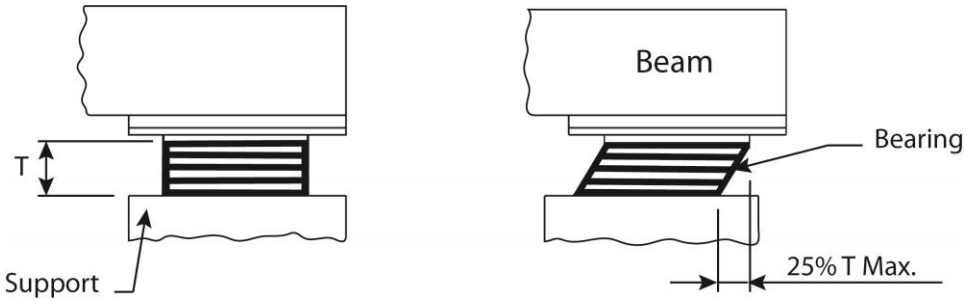

Figure 12.7 Schematic of an Elastomeric Bearing

12.1.3 High Load Multi-Rotational

12.1.3.1 Pot Bearings

Pot bearings generally consist of a masonry plate, steel pot plate which contains an internal elastomeric disc, a steel piston, and a layer of PTFE covered by a steel slide plate, as shown in the schematic in Figure 12.8. Pot bearings are generally used in situations where there are large loadings and rotations occurring in multiple/varying directions (i.e., large spans, deep heavy girders, curved bridges, skewed bridges, etc.). An example of a pot bearing is shown in Figure 12.9.

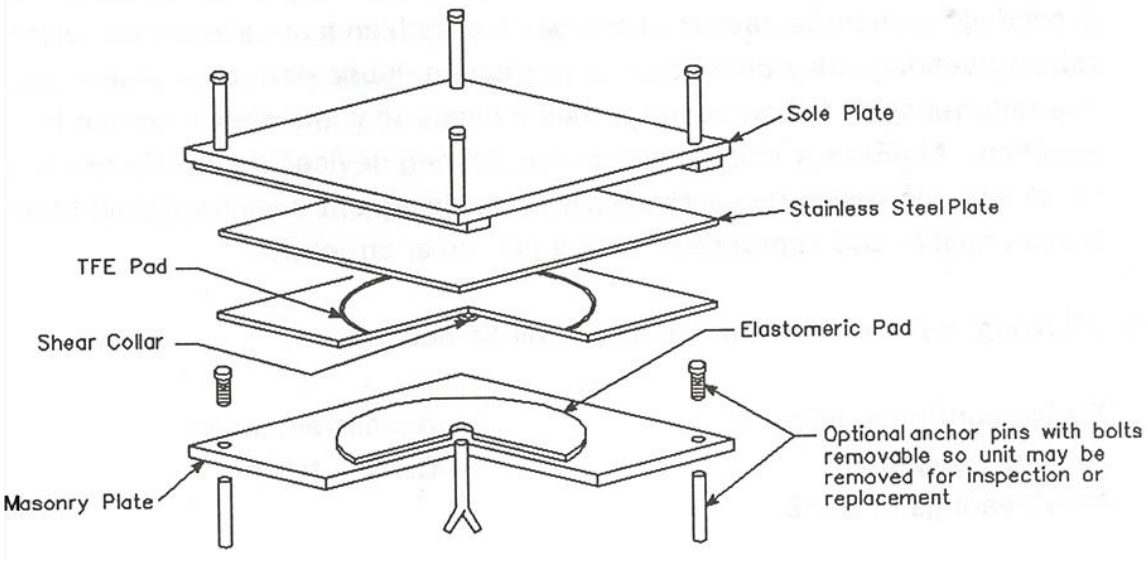
Sole Plate
Stainless Steel Plate
TFE Pad
Shear Collar
Elastomeric Pad
Optional anchor pins with bolts removable so unit may be removed for inspection or replacement
Masonry Plate

Figure 12.8 Schematic of Pot Bearing

Figure 12.9 Example of a Pot Bearing

12.1.3.2 Spherical Bearings

Spherical bearings generally consist of a concave bottom steel plate which may or may not contain a layer of PTFE within the machined surface and also a convex steel plate which is connected by a sole plate. Spherical bearings are typically used when there are very large loadings which also require rotations about any axis. The spherical bearing is described as a ball and socket which provides little resistance to rotation.

12.1.4 Specialty Bearings

12.1.4.1 Seismic Isolation Bearings

The purpose of seismic isolation bearings is to dampen the forces transmitted from the substructure to the superstructure during a seismic event. These bearings may appear to have configurations as other bearing previously described, but contain internal materials which

cannot be inspected or maintained. Seismic isolation bearings can come in a variety of configurations and are typically proprietary to the fabricator of the bearing. An example of a seismic isolation bearing is shown in Figure 12.10.

Figure 12.10 Seismic Isolation Bearing

12.2 Preventive and Basic Maintenance of Bearings

Preventive and basic maintenance of bearings could sustain the design life of a bridge and/or prevent a bridge failure from occurring.

12.2.1 Cleaning

Removing foreign material such as dirt, debris, vegetation and pack rust will help the performance of the bearing since these items could cause the bearing to not function properly. These materials for instance could cause a sliding bearing to freeze, or a rocker bearing to not rotate properly if these materials are not removed from the area. Additionally, progressive ratcheting of rocker bearings can occur from accumulation of additional debris under one side of the bearing during each extreme temperature season, causing the bearing to be unable to rotate back to original position. This has led to bearings tipping over.

This task should be performed on routine intervals.

12.2.2 Anchor Bolt Tightening

Bearings should be inspected for loose anchor bolts. If during an inspection a bearing is found to have loose anchor bolts, they should be tensioned properly especially for a fixed bearing assembly, and even for an expansion bearing if uplift is a concern. If a bearing assembly has loose anchor bolts, the bearing may not be functioning properly and may be causing stress to the superstructure or substructure, as well as to the bearing, which may not be visible. If anchor bolts are not properly torqued, uplift may also be occurring.

12.2.3 Power Washing

Power washing of steel bearings which are exposed to corrosive chloride environments should help in maintaining integrity of the bearing protective coating. This is especially important for bearings at expansion joints where water and salts melt from the roadway above and leak onto the bearing area. Power washing of these locations should be considered a priority once the winter season is over.

12.2.4 Lubrication of Sliding and Rotating Surfaces

Surfaces which are meant to allow for sliding and rotation should be cleaned and lubricated as needed to ensure the bearing is functioning properly and is clean of debris, unless access to the bearing is not achievable. An example of bearing lubrication is shown in Figure 12.11. Power washing or compressed air can be used to clean the bearings. Lubrication can utilize grease, oil, or graphite or other product deemed satisfactory by the owner.

Figure 12.11 Bearing Lubrication

12.3 Repair and Rehabilitation of Bearings

Bearings that are not functioning as they were designed, such as a frozen expansion bearing, may overstress beams and supporting substructure or in the extreme case cause a bridge failure. If bearings are found not to be functioning as designed, they shall either be repaired or rehabilitated to match or exceed the original design intent.

12.3.1 Replacement vs. Repair

When to Call the Engineer

Call the Engineer when determining if bearings should be repaired or replaced.

If it is determined that action must be taken for a bridge bearing to function properly, replacement of the bearing or repair of the bearing should be evaluated. A few key questions should be asked during the evaluation process:

- Is the bearing stressed beyond the point of repair?
- If repaired, could it possibly require future repairs prior to the end of the expected life of the bridge?
- Is there a better option to improve the function of the bridge?
- Is it cheaper to replace than repair (thinking about future costs)?

It is obviously at the owner's discretion how to proceed, however answers to questions such as these can help an owner make an informed decision. Every circumstance will be a case by case basis, but replacement may be required to yield a long term economical solution that increases the life of the structure. If a bearing simply needs to be reset, the replacement cost may not be justified.

12.3.2 Jacking of the Superstructure

When to Call the Engineer

Call the Engineer before planning jacking activities.

Replacing or repairing bridge bearings often requires the superstructure to be lifted or "jacked." It is important to understand that jacking the superstructure is a potentially dangerous activity that could result in a bridge collapse, serious structural damage, injury to workers or a traffic accident. All jacking activities should be thoroughly planned by an engineer prior to being performed.

When jacking the superstructure, some general guidelines should be followed:

- A jacking procedure should be put together by the agency/contractor performing the work and should be reviewed by a licensed engineer prior to commencing.
- Lifting points shall be placed under the main carrying beams of the structure unless a licensed engineer has determined that the diaphragm connection has sufficient capacity to be utilized in lifting the superstructure.
- All beams in the cross section shall be lifted with the same amount of force to avoid excessive stress to the bridge.
- Lifting points shall be carefully considered and planned out prior to jacking the superstructure. Bearing lines of support should always be jacked to the same lift height. If the bridge is a simple span structure, each end of the structure can be lifted independent of the other. If the bridge is a continuous span structure, it should be evaluated by an Engineer to determine if the bridge type, span lengths, and jacking heights would require multiple lines of support to be simultaneously lifted.
- Jacking force shall be determined by calculating the weight of the structure at each particular lifting point and then multiplied by a factor of two or as otherwise determined by the Engineer
- During the jacking process, live load should not be allowed on the bridge unless the jacking forces have accounted for live load plus impact. If live load is allowed during the jacking process, the fluctuating forces being applied to the jacks may need to be considered. Both vertical and horizontal live load forces should be considered.
- As the structure is being jacked, temporary blocking should be placed adjacent to the jack in case the jack fails during the lifting process.

- Depending on the required height the structure needs to be jacked, temporary transitions over deck joints may need to be installed in order for traffic to smoothly transition on and off the bridge.
- If there is insufficient space at the point of support for jacking to be performed, a temporary shoring tower or bent can be constructed adjacent to that point of support.
- The amount of time the structure is lifted by the hydraulic jacks alone shall be determined by the manufacturer of the jacks. If the structure needs to be lifted for longer than the hydraulic jacks allow, the structure shall be lifted by the jacks and placed on temporary blocking until the work is completed.
- Prior to jacking, all bearing anchor bolt nuts shall be removed, bridge railing shall be disconnected (if applicable), and/or impacts to utilities on the bridge (if applicable) should be considered.
- Once the superstructure is jacked (see Figure 12.12), it should be periodically checked for settlement.
- Prior to lowering the superstructure, ensure that positions are correct and anchor bolts are properly aligned with holes/slots.
- After the superstructure is lowered, ensure that deck joints line up and are functioning properly.

Figure 12.12 Lifted or "Jacked" Superstructure

In some circumstances, jacking the bridge from underneath is not a feasible option for many reasons including clearance of existing roadway beneath, bridge is located high above water, bridge is over a railroad, etc. In these instances, the bridge can be jacked above the deck by means of a carrier beam. A carrier beam can be installed above the deck on timber blocking behind the existing support locations. Once the carrier beam is in place, support brackets can be connected to the existing beam by installing threaded rods through the deck. Once all supports are in place, the ends of the carrier beam will be jacked from behind the blocking, which then lifts the superstructure up off its bearings. A schematic of the carrier beam method for bridge jacking is shown in Figure 12.13.

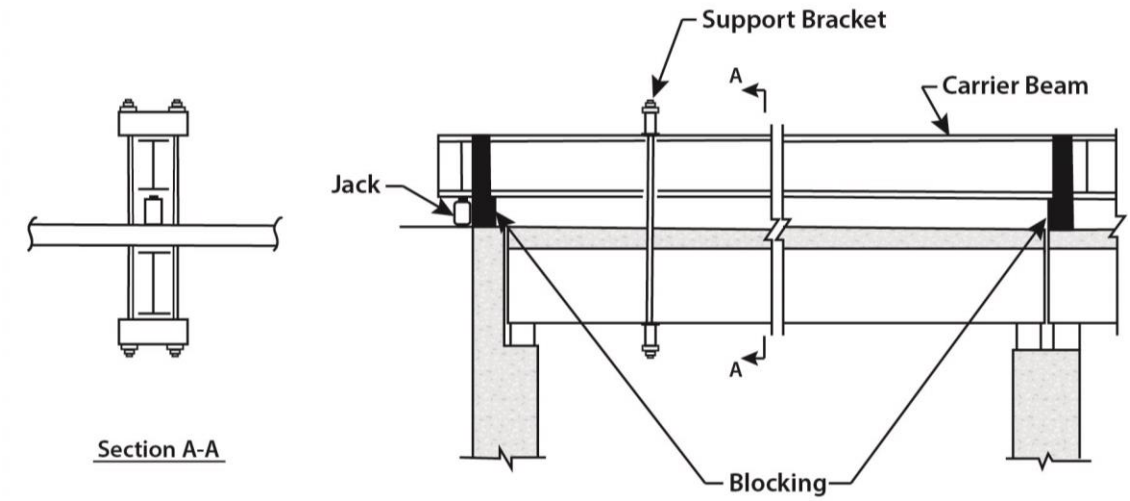

Figure 12.13 Carrier Beam Method for Bridge Jacking

Some of the drawbacks to using a carrier beam system include:

- It will limit space on the bridge deck for traffic and requires traffic control systems.
- It could require fabrication of a beam unless a salvaged beam is available with sufficient capacity.
- The top flange of the carrier beam may need to be braced for buckling resistance which would also limit space on the topside of the deck.
- It may only lift the bridge in segments rather than uniformly across the cross section.
- It requires holes to be drilled through the deck.

If it is determined necessary to use a carrier beam, it shall be designed by a licensed engineer to withstand all the necessary dead loads and live loads to which it will be subjected.

When to Call the Engineer

The carrier beam should be designed by a licensed engineer to withstand all the necessary dead loads and live loads to which it will be subjected.

12.3.3 Resetting of Bearings

If bearings require resetting, they should be positioned so that they are centered at the median temperature for the geographical area. If bearings need to be set or reset at temperatures other than median temperature, the bearings can be offset by using the (contributing expansion length in inches) multiplied by (coefficient of expansion) multiplied by (± delta degree from the median temperature). For steel superstructures, the coefficient of expansion is 0.0000065 (6.5×10^{-6}) per °F. For concrete superstructures, the coefficient of expansion is 0.0000055 (5.5×10^{-6}) per °F. An example is shown in Figure 12.14. The contributing expansion length can be the span length for a simple span but can be different for a continuous span. An engineer should be consulted.

Figure 12.14 Resetting Bearings

12.3.4 Replacement of Bent or Sheared Anchor Bolts

Anchor bolts that have been inspected and found to be bent or sheared off should be replaced.

Suggested Procedure

Replacement of Bent or Sheared Anchor Bolts

1. Put together a replacement plan and jacking procedure (See 12.3.2 Jacking of the Superstructure).
2. Refer to existing construction drawings (if available) and perform measurements to determine where the existing anchor bolts are located. Place new anchor bolts at least 2 inches from existing anchor bolt locations.
3. Fabricate required replacement parts (if necessary, masonry plates, bearings, sole plates, etc.)
4. Install traffic control devices (if necessary) and secure the work zone
5. Jack the existing superstructure, as described in 12.3.2 Jacking of the Superstructure.
6. Remove old bearing assemblies, clean off existing substrate of all debris, mark out locations of new anchor bolts, drill anchor bolt holes, install and grout new anchor bolts as required by design.
7. Install existing or new bearing assemblies
8. Lower the superstructure back into proper position
9. Remove jacking and traffic control devices

When bearings have broken or sheared anchor bolts, all bearings on the bridge should be inspected, as a bent or broken anchor bolt indicates that bearings/joints are not allowing enough movement, and the resulting forces from the restraint are damaging the anchor bolts.

12.3.5 Installation of Bearing Restraints

If during inspection it is recognized that the bridge bearings are “walking” or have shifted from their original design location, restraints can be installed to ensure the beams do not fall off the bearings or that the bearings are not damaged by misalignment. Bearing restraints generally consist of a plate that fits around the existing bearing with anchor bolts to hold it in place. An example of a restrainer plate is shown Figure 12.15.

Figure 12.15 Restrainer Plate

An Engineer must evaluate restrainer plate thickness to bearing depth to determine if the plates limit translation of the bearing. An alternative to restrainer plates would be to replace the bearings with a non-wax impregnated elastomer pad with adhesive attachment. By not having wax in the elastomer material, the tendency for the bearing to walk out is lower, as is the use of adhesive. If restrainer plates are acceptable to the Engineer, the following procedure should be followed:

Suggested Procedure

Installation of Bearing Restraints

1. Refer to existing construction drawings (if available) and perform measurements to confirm the existing bearing dimensions.
2. Fabricate required bearing restraint plates and anchor bolts.
3. Install traffic control devices (if necessary) and secure the work zone.
4. Jack the existing superstructure, as described in 12.3.2 Jacking of the Superstructure.
5. Remove and clean off existing bearings and substrate.
6. Drill anchor bolt holes, install and grout new anchor bolts.
7. Install existing bearings and restraints.
8. Lower the superstructure back into proper position.
9. Remove jacking and traffic control devices.

12.3.6 Blast Cleaning and Coating of Corroded Steel Bearings

If existing steel bearings are found to be in good condition with the exception that the protective coating is failing or has failed, the following procedure can be used to rehabilitate the bearings.

Suggested Procedure

Blast Cleaning and Coating of Corroded Steel Bearings

1. Install traffic control devices (if necessary) and secure the work zone.
2. Jack the existing superstructure, as described in 12.3.2 Jacking of the Superstructure.
3. Remove the existing bearings and clean off the substrate of all debris.
4. Blast clean the existing bearings in all areas that require recoating (if a sliding bearing, sliding surfaces should be cleaned and lubricated) .
5. The bearing shall then be prepared and painted in accordance with the owners/agencies current standards for such repairs.
6. Re-install the bearing device.
7. Lower the superstructure back into proper position.
8. Remove jacking and traffic control devices.

Note that if the coating is only failing in localized areas, attempt should be made to remove deterioration and repaint those localized areas. Another option of repairing localized areas of deterioration is to treat these areas with a spray applied product that removes the rust and simultaneously creates a protective coating. This protective coating would then prevent future deterioration. Both of these options will greatly reduce the scope of work since jacking the superstructure and complete removal of the bearings would not be required.

12.4 Gusset Plates

12.4.1 Introduction

What To Look For

- Accumulated debris
- Section loss/corrosion
- Cracks
- Deformation or buckling
- Missing or damaged bolts or rivets

Gusset plates are truss connecting elements that are located at the truss panel points. They connect truss chords, verticals, diagonals, and bracing members together. As such, they are a major load transfer element of the truss carrying significant loads and stresses.

Failure of gusset plates was determined to be a major contributing factor leading to the I-35W Mississippi River Bridge collapse that occurred in August 2007. Following this catastrophic event, the design load rating and inspection criteria for gusset plates were re-evaluated and updated nationwide.

Gusset plates are typically designated and identified based on their orientation and location, as indicated in Figure 12.16 below. Examples of gusset plate orientation and location include horizontal (lateral) and vertical (either internal or external).

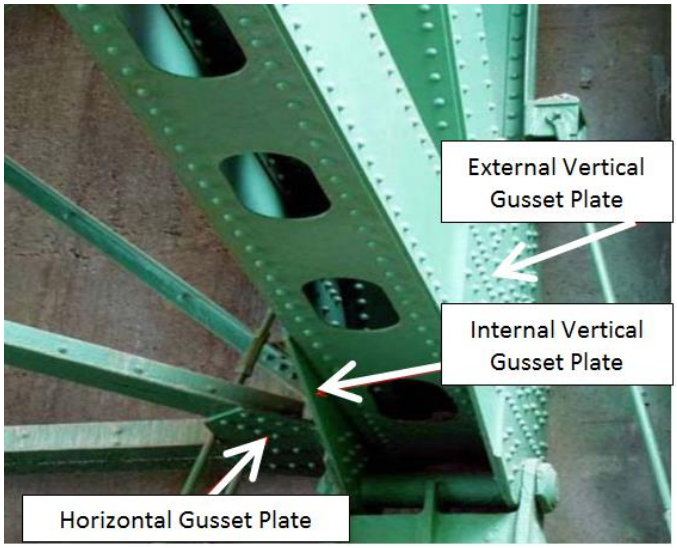

Figure 12.16 Examples of Gusset Plate Orientations and Locations

The truss member connections are susceptible to trapping debris. If not cleaned, the situation may lead to failure of the coating system allowing corrosion at the interface of the gusset plate and connecting members. Common locations of gusset plate detrioration are atop of horizontal gusset plates where gusset plates meet horizontal truss members, and inside the truss panel points. (The area that is semi enclosed by gusset plates and truss members).

12.4.2 Major Repairs, Strengthening or Replacement of Gusset Plates

Major repair, strengthening or replacement of gusset plates is complex and should be performed only by a qualified contractor or agency staff. Because gusset plates are critical to the structural integrity of the truss, repairs should not be attempted by inexperienced or unqualified personnel. Before any work on gusset plates is undertaken, the truss should be analyzed by a structural engineer to develop the necessary bracing, shoring, jacking, and installation procedures specific to the gusset plate being repaired. Failure to carefully follow the bracing, shoring, jacking and installation procedures could result in failure of the truss.

When to Call the Engineer

Contact the Engineer before working on a gusset plate.

12.4.3 Minor Repairs and Strengthening of Gusset Plates

Minor repairs and strengthening of gusset plates require much less effort and can be effectively performed by a bridge maintenance personnel or experienced ironworkers.

These repairs may include:

- Replacement of rivets with HS bolts to increase the connection capacity
- Stiffening of unsupported edges to prevent buckling
- Adding a reinforcement plate at localized section loss locations or for strengthening
- Replacement of horizontal (lateral) gusset plates connecting secondary members
- Other minor repairs that are not covered in this chapter may also be performed (e.g., replacement of lacing bars, end batten plates, internal diaphragms/bulkheads, etc.)

12.4.3.1 Replacement of Rivets with High Strength Bolts to Increase Connection Capacity

If structural analysis determines that a stronger connection is needed, and the rivets control the strength of the connection, strengthening can be achieved by replacing the existing rivets with High Strength (HS) bolts of the same or larger diameter. HS bolts are typically at least 10 percent stronger than rivets. Unless a structural analysis determines otherwise, no more than one or two rivets should be removed and replaced at a time. Rivet removal and replacement method and procedures were provided in Section 11.5. An example of rivet removal and replacement on a gusset plate is shown in Figure 12.17.

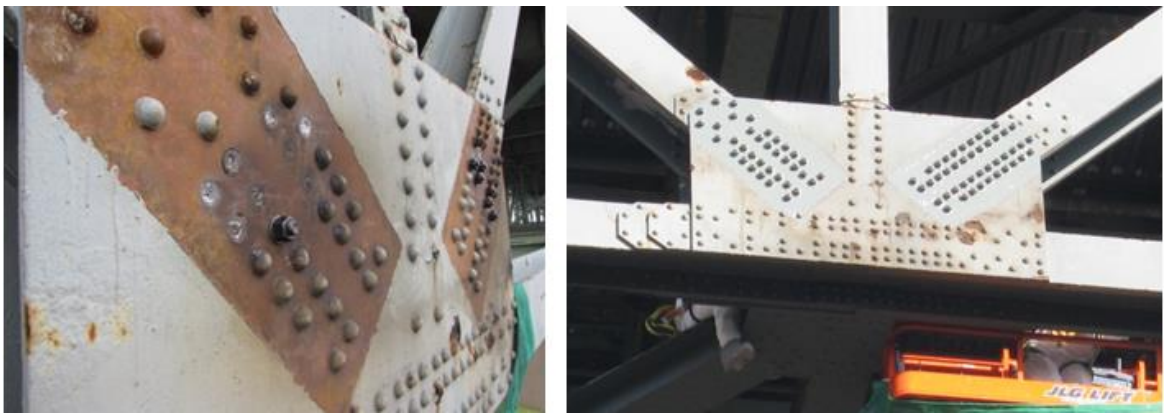

Figure 12.17 Example of Gusset Plate Repair. Rivet Removal (left) and Replacement with HS Bolts (right)

12.4.3.2 Stiffening of Unsupported Edge to Repair or Prevent Buckling

Truss connections can be very stiff but there are typically portions of gusset plates that are unsupported and may tend to bow or buckle when loaded. An example is shown in Figure 12.17. Engineers have been developing sophisticated analysis approaches to evaluate the existing buckling resistance in these connections. There used to be general guidelines based on the length of the unsupported edge as compared to the yield strength of the steel to help safety inspectors determine if the plates may be undersized. But that approach was too simplistic. The important thing for a bridge maintenance worker to remember when working in

and around gusset plates is that these unsupported edges may buckle, and if any such bending such as shown in Figure 12.17 is noticed, it should immediately be reported to any engineer to determine the cause and if any strengthening is required.

When to Call the Engineer

Call the Engineer if bending or buckling of the gusset plates is observed, to determine the cause and if additional strengthening is required.

Figure 12.18 Example of Gusset Plate Buckling

A stiffening angle is often added to prevent or repair buckling. The angle selected should have a leg thickness greater than or equal to the gusset plate thickness. 7/8 inch diameter A-325 HS bolts spaced at 3 inches to 6 inches are typically used to attach the stiffening angle. Examples of this repair are presented in Figure 12.19.

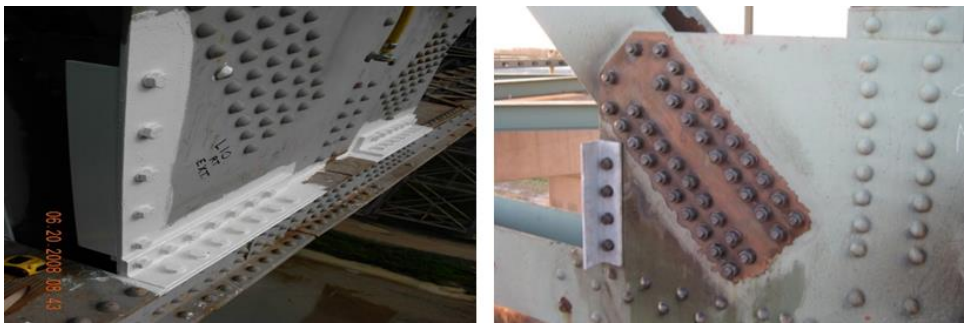

Figure 12.19 Examples of Stiffening Plates Added to Gusset Plate Edges

When to Call the Engineer

All repair procedures should be developed or approved by a qualified engineer.

12.4.3.3 Reinforcement at Localized Section Loss Locations

Surface pitting and localized section loss of the gusset plate surfaces can be repaired by installation of a reinforcement plate to cover and reinforce the affected area as shown in Figure 12.20.

Figure 12.20 Gusset Plate Repair by Installation of Reinforcement Plate

Suggested Procedure

Gusset Plate Repair

1. Fabricate the reinforcement plate with holes drilled to match the existing rivet pattern. The reinforcement plate should be the same grade of steel, and have a thickness greater than or equal to the existing gusset plate.
2. Thoroughly clean the repair area, removing all loose paint and oxidized metal. The cleaning should be performed utilizing needle scalers, power wire brush, grinder, or sandblaster. If grinding, do not gouge which could later cause crack initiation.
3. Paint the repair area with a zinc based primer.
4. Remove rivet heads and verify fit of the reinforcement plate.
5. Drive first rivet shank out and attach reinforcement plate with HS bolt of the same diameter. The bolt shall be tightened snug tight.
6. Remove second rivet shank, preferably located at the diagonally opposite location, and install HS bolt.
7. Verify that the remaining bolts will fit.
8. Install remaining bolts one at a time.
9. Initially snug tighten all the bolts making sure all surfaces are mated. Use turn-of-nut method to fully tension all the bolts after the snug tight operation, or use tension-controlled bolts. See turn-of-nut requirements published by the Research Council on Structural Connections and Chapter 13 of this manual.
10. Paint the repair with a paint system to match the existing paint.

A similar procedure can be used for a gusset plate for connection strengthening.

12.4.3.4 Replacement of Horizontal (Lateral) Gusset Plates

These repairs are typically performed for secondary truss members such as lateral bracing, and sway frame members. The repair may require bracing to stabilize the existing members prior to disconnecting them from the gusset plate and should be completed under the guidance of an Engineer following development of a location specific repair procedure.

Typically the following order of repair operations should be followed:

1. Install support or bracing in accordance with the repair procedure. Tension ties can be used for tension members and compression struts for members that are in compression. Sometimes support from other truss members can be effective, as can be seen in Figure 12.21.
2. Make a template of the existing gusset plate; fabricate replacement plate and connecting elements.
3. Disconnect bracing member from the deteriorated gusset plate.
4. Remove existing gusset plate.
5. Clean the repair area; remove all loose paint and oxidized metal utilizing needle scaler, power wire brush, grinder, or sandblaster. If grinding, do not gouge which could later cause crack initiation.
6. Paint the repair area with a zinc based primer.
7. Install replacement gusset plate.
8. Reconnect previously disconnected bracing member using high strength bolts.
9. Paint the repair with a paint system matching existing.

Figure 12.21 End Diagonal Support

The actual step-by-step repair procedure shall be developed based on repair specifics and actual field conditions.

Before and after repair photos of a vertical gusset plate replacement are shown in Figure 12.22.

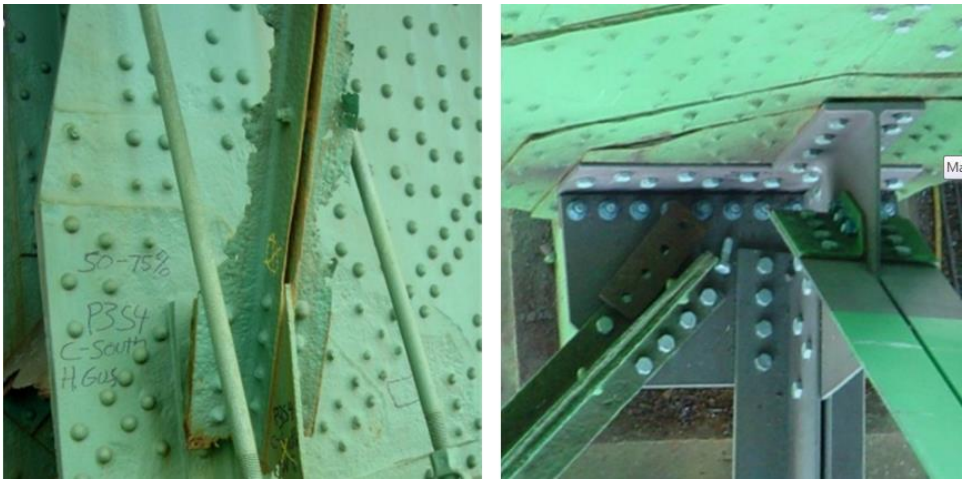

Figure 12.22 Lateral Gusset Plate Repair. Before Repair (left). After Repair (right).

12.5 Cables

12.5.1 Introduction

There are essentially three types of bridges that use the cable system:

- Cable Stayed Bridges
- Suspension Bridges
- Tied Arch Bridges

12.5.2 Cable Types

Regardless of the bridge design and type, these are the types of cables generally available:

- Cables of locked coil construction
- Cables with parallel wires
- Cables with parallel strands
- Parallel-bar Cables

12.5.2.1 Locked-Coil Cables

In the locked-coil cables, the wires are arranged in successive layers, wound around a central core which consists of circular, parallel wires (Figure 12.23). On the outside, elongated S-sections are used, and in view of the extent of their overlapping, they form an envelope which is more or less watertight (hence the name 'locked-coil cable'). The advantages of locked-coil cables are the ease of placement, the economy (because ducts and grouting are unnecessary), the reduced anchorage space, and the great flexibility which makes it possible to use guiding saddles at the pylons instead of intermediate anchorages.

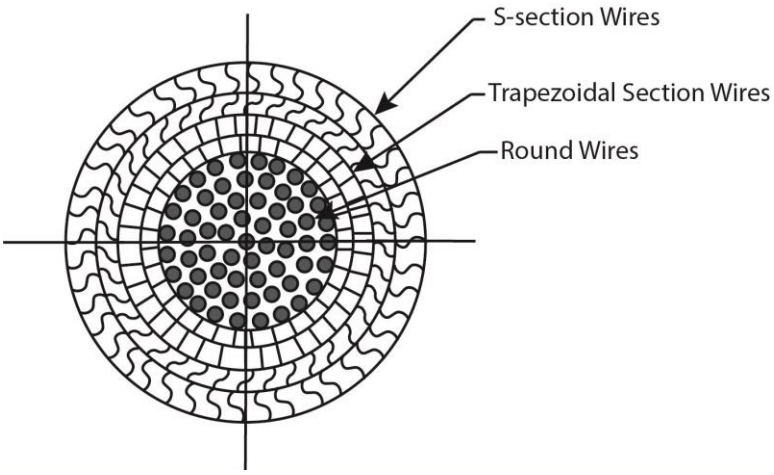

Figure 12.23 Locked-Coil Cable

12.5.2.2 Cables with Parallel Wires

These are of high-strength, drawn steel that are placed in metal or polyethylene ducts. Parallel wire cables used in cable-stayed bridges conforms to ASTM A421, type BA, low relaxation. Corrosion protection is provided by the polyethylene sheathing filled with cement grout (Figure 12.24). The tubing is usually wrapped with polyvinyl film tape.

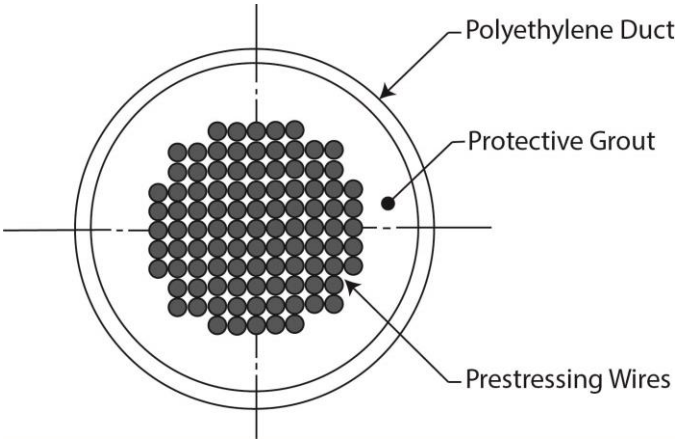

Figure 12.24 Parallel Wire Cable

12.5.2.3 Cables with Parallel Strands

Structural wire strand is an assembly of wires formed helically around a center wire in one or more symmetrical layers. A schematic of a cable with parallel stands is shown in Figure 12.25. Cable sizes normally range from 50 to 100 mm.

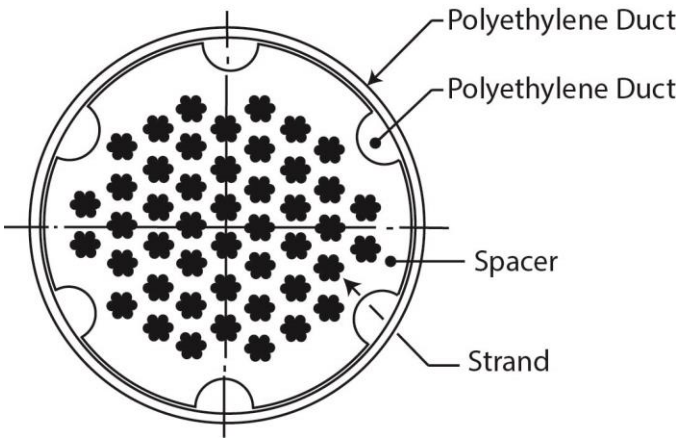

Figure 12.25 Parallel Wire Strand

When the cables are stressed, the resultant lateral stresses reduce the fatigue resistance. Also, sensitivity to corrosion is increased, since, for a given cross-sectional area the perimeter of a section made up of many wires is greater than that of a single circular member.

12.5.2.4 Parallel Bar Cables

Parallel bar cables are formed of steel rods or bars which are parallel to each other and encased in metal ducts kept in position by polyethylene spacers. Examples of parallel bar cable cross-sections, with and without couplers, are shown in Figure 12.26. The bars can slide in the longitudinal direction, which simplifies the process of tensioning them individually. Injection of cement grout, carried out after erection, enables the system to act as one unit in resisting stresses.

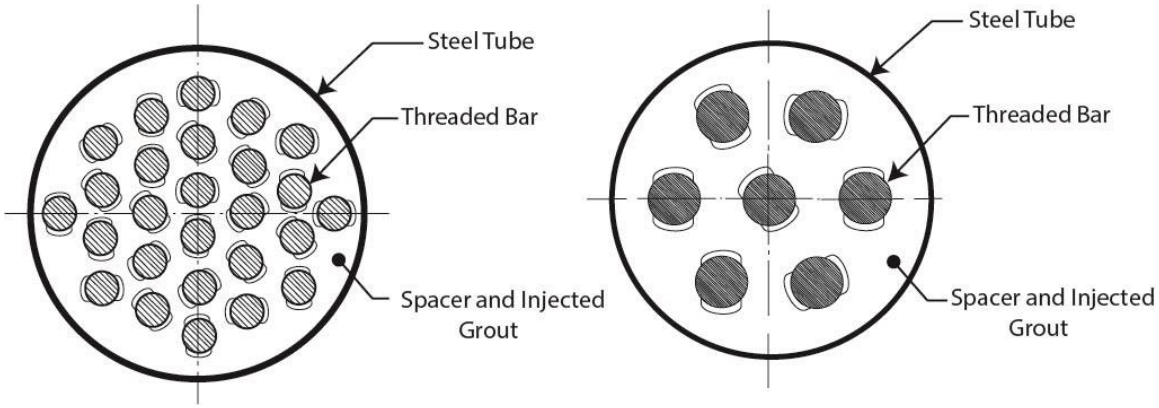

Figure 12.26 Parallel Bar Cables. Without Couplers (left) and with Couplers (right)

12.5.3 Suspension Bridge Components

The main suspension cables in a suspension bridge are typically supported on saddles at the top of the towers and are anchored at each end, as shown in Figure 12.27. The suspension cables are in tension and require massive anchorage at both ends, and are typically load-path non redundant. The main cables are usually of parallel wires clamped at points where suspenders connect with them to support the bridge deck. The individual wires have a 0.192 inch diameter,

and a 0.002 inch zinc coating around the wire, resulting in a total diameter of 0.196 inch. The zinc coating provides cathodic protection against corrosion. The suspension bridge cables transfer loads to the towers via the saddles atop the towers, the performance of the cables is affected by the saddle detail.

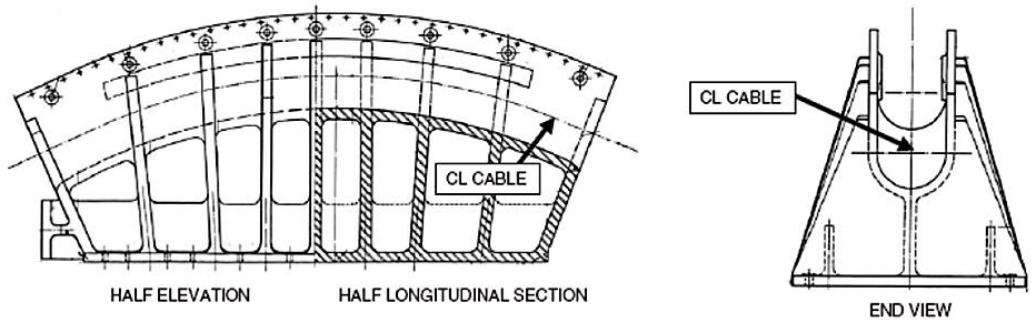

Figure 12.27 Elevation (left) and Cross-Section View of a Typical Saddle (right)

Suspender cables are the vertical cable elements that connect the deck sections to the main cables. The composition of the suspender cables varies considerably. Suspenders made of wire rope have two legs, but they are actually a single continuous piece that rises from the deck, loops over the cable band, and returns to the deck, using only two sockets. Examples of suspender cables are shown in Figure 12.28. When suspenders are designed to ride cross saddle over the cable bands, then they contribute to the normal force provided by the cable band bolts and increase resistance to cable sliding. If suspenders are made of wire strands that cannot be bent over the main cable, then they require a socket, as shown in Figure 12.28 (right). They are much stronger and thus lighter than rope suspenders and consequently simplify cable band detailing.

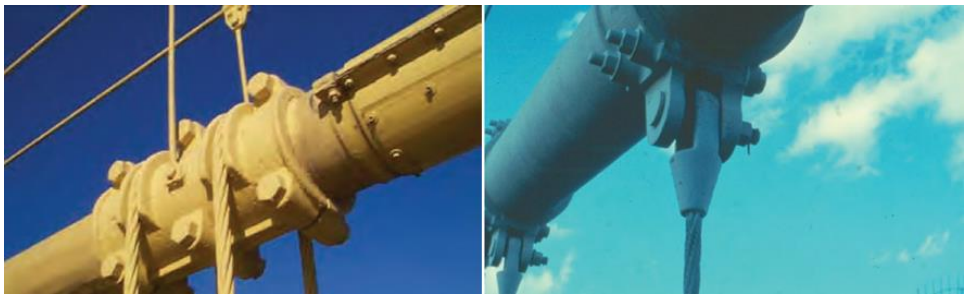

Figure 12.28 Suspender Cable Looped over Cable Band (left) and Suspender Cable with Open Socket Connection (right).

Cable bands consist of two cylinder halves bolted together over the circumference of the cable (Figure 12.29). The number of bolts per cable band is dependent on the slope of the cable at the suspender attachment points. The tension in the cable band bolt that squeezes the two halves together decreases with time due to creep of the zinc coating on the wires.

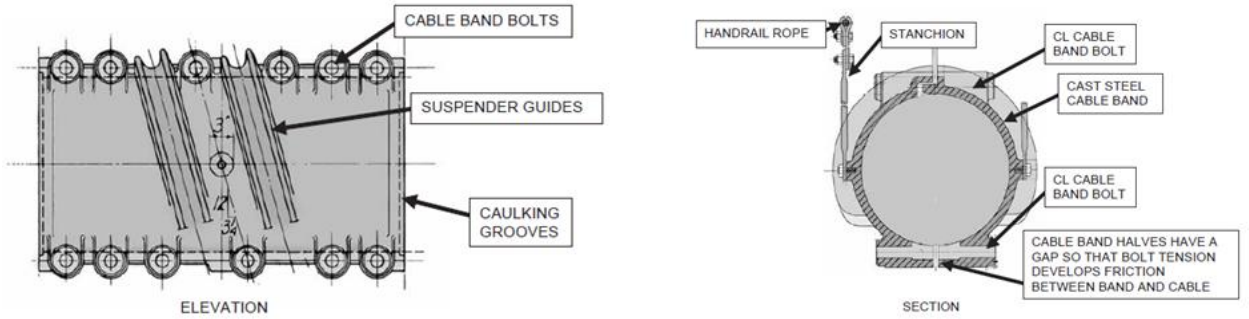

Figure 12.29 Cable Band Schematic. Elevation (left) and Section (right) Views.

12.5.4 Cable Stay Bridges

The concept of a cable-stayed bridge is simple. A bridge carries mainly vertical loads acting on the girder. The stay cables provide intermediate support for the girder so that it can span a long distance. The basic structural form of a cable stayed bridge is a series of overlapping triangles comprising the pylon, the cables and the girder. All these members are under predominantly axial forces, with the cables under tension and both the pylon and the girder under compression.

Cables are the most important elements of a cable stayed bridge. They carry the load of the girder and transfer it to the tower and the back stay cable anchorage. The cables in a cable-stayed bridge are all inclined. The most popular type of cable nowadays uses seven wire strands. These strands were originally developed for prestressed concrete applications and offer good workability and economy. They can be either shop fabricated or site-fabricated. In most cases, corrosion protection is provided by a high-density polyethylene pipe filled with cement grout.

As an alternative to cement grout, greased or galvanized strands of the cables could be individually sheathed. A sheathed, galvanized strand may have wax or grease inside the sheathing.

12.5.5 Tied Arch Bridges

The tied arch bridge is the other bridge type that relies on hangers of the cable type as part of its load supporting system. The hangers are mostly in tension and are vertically connected to the deck. Some hangers cross other hangers at least twice in which case the bridge is known as network arch bridge.

12.5.6 Cable Defects

What To Look For

- Corrosion
- Pitting
- Section Loss
- Failure of the Paint System
- Collision Damage
- Overload Damage
- Fatigue Cracking

12.5.7 Inspection Locations and Procedures

Cable supported bridges are unique in their design and behavior. Typically, newer Cable Stayed Bridges and Suspension Bridges have an individualized maintenance manual akin to an owner's manual. Older and less complex wire rope or eyebar suspension bridges may not have such a manual. When available, it should be first resource of the inspector to gain understanding of the material type of the cable and the design protocol. While the procedures enumerated below are not necessarily exhaustive, they can serve as a guideline to a satisfactory inspection regime in conjunction with the maintenance manual if available. However, the ability to recognize potential issues in between the regular biennial inspections is very important.

12.5.7.1 Main Suspension Cable

Main Suspension Cables

Inspect the main suspension cables for indications of corroded wires. Inspect the condition of the protective covering or coating especially at low points of cables, areas adjacent to the cable bands, saddles over towers and at anchorages.

Figure 12.30 Main Cable Inspection

Cable Wrappings

Inspect the wrapping wire for cracks, staining and dark spots. Check for loose wrapping wires. If there are cracks in the caulking where water can enter, this can cause corrosion of the main suspension cable. Check for evidence of water seepage at the cable bands, saddles, and splay casting saddles. Note that the force from the main cable is distributed through the splay saddle, bridge wires, strand shoes, and anchor bars. These items are shown in the anchor block schematic in Figure 12.31.

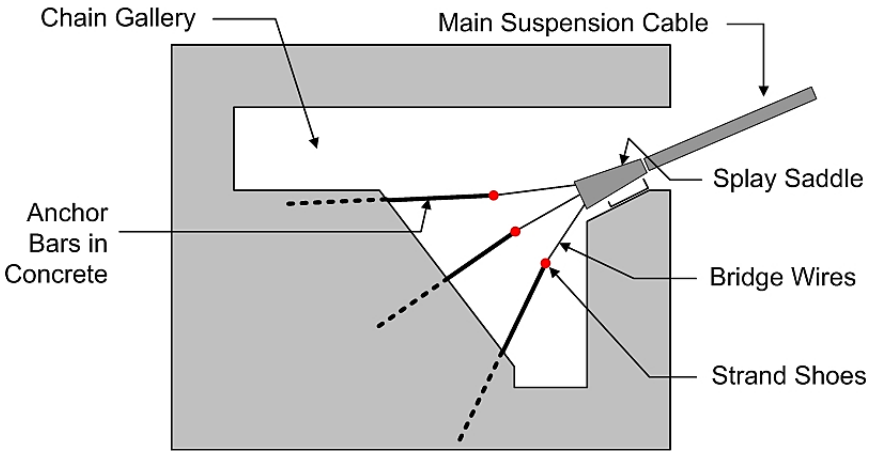

Figure 12.31 Anchor Block Schematic

Cable Bands

Inspect the cable bands for missing or loose bolts or broken suspender saddles. Signs of possible slippage are caulking that has pulled away from the casting or “bunching up” of the

soft wire wrapping adjacent to the band. Check for the presence of cracks in the band itself, corrosion or deterioration of the band, and loose wrapping wires at the band.

Sockets

Inspect the suspender rope sockets for corrosion cracks or deterioration

- Abrasion at connection to bridge superstructure
- Possible Movement

Suspender Cables and Connections

Inspect the suspender cables for corrosion or deterioration, broken wires and kinks or slack. Check for abrasion or wear at sockets, saddles, clamps and spreaders. Note any excessive vibrations.

Vibrations

Note and record all excessive vibrations.

Hand Ropes

Inspect the hand ropes and connections along the main cables for loose connections of stanchion to cable bands or loose connections at anchorages or towers. Check also for corroded or deteriorated ropes or stanchions, bent or twisted stanchions (hand rope supports), and too much slack in rope.

12.5.7.2 Cable Stay Bridge Inspection

The inspection of the cable elements should include the following:

- Cable wrappings and wrap ends near the tower and deck
- Cable sheathing assembly
- Dampers
- Anchorages

An example of stay cable inspection is shown in Figure 12.32.

Figure 12.32 Inspection of Stay Cables

Cable Wrapping

Common wrapping methods for corrosion protection of finished cables include spirally wound soft galvanized wire, neoprene, or plastic wrap type tape. The wrappings should be inspected for corrosion and cracking of soft galvanized wire, staining, and dark spots indicating possible corrosion of the cables and loose wrapping wire or tape.

Check for evidence of water seepage at the cable bands, saddles and splay castings.

Cable Sheathing Assembly

The most common types of cable sheathing assemblies are steel sheathing and polyethylene sheathing.

If steel sheathing is used, inspect the system for corrosion of protective coatings and weld fusion issues. Bulging may indicate broken wires. An example of bulging is shown in Figure 12.33. Splitting may be caused by water infiltration and corrosive action. Cracking is sometimes caused by fatigue. An example of deterioration of steel sheathing is shown in Figure 12.33.

If polyethylene sheathing is used, inspect the system for nicks, cuts, and abrasions.

Check for cracks and separations in caulking and in fusion welds. Bulging may indicate broken wires. Splitting is sometimes caused by temperature fluctuations because the coefficient of the thermal expansion for polyethylene is three times higher than the value for steel or concrete. Cracking is sometimes caused by fatigue.

Figure 12.33 Example of Steel Sheathing Deterioration. Corrosion (left) and Bulging (right).

Maintenance worker repair of any cracking or splitting of the sheathing should only be temporary in nature until an engineered solution can be determined.

When to Call the Engineer

If cracking, splitting, or bulging of cable sheathing is observed, it may indicate more significant distress or damage of the cable inside.

Anchorages

Inspect the transition area between the steel anchor pipe and cable for water tightness of neoprene boots at the upper ends of the steel guide pipes. Check for drainage between the guide pipe and transition pipe, and defects, such as splits and tears, in the neoprene boots. Check for sufficient clearance between the anchor pipe and cable, noting rub marks and kinks.

Tower Anchorage

Inspect the cable anchorages for corrosion of the anchor system. Check for cracks and nut rotation at the socket and bearing plate, and seepage of grease from the protective hood.

Other Items

For example, the inspection of the cable system should also include anchor pipe clearances, flange joints, and polyethylene expansion joints. Note and record all excessive vibrations including amplitude and type of vibration along with wind speed and direction.

12.5.8 Cable System Maintenance

In this segment we consider some of the techniques and methods available for maintaining the aforementioned bridge cable systems once an inspection has been completed.

12.5.8.1 Cable Compaction

Cable compaction may be required of the main cable of a suspension bridge. Cable recompaction starts at one of the cable bands and proceeds towards the other cable band. The compactor normally consists of a segmented steel ring with four 100 ton center hole hydraulic

jacks operating simultaneously to constrict the ring. The jacks were equally pressurized through a manifold and a hydraulic pump powered by compressed air. Compaction intervals and temporary seizing band spacing are determined in the field based on the degree of expansion of the cable after wedging. Cable circumference measurements are taken after pressurizing the compactor and after banding and releasing the compactor to monitor the relaxation of the bands and to ensure that the original diameter was not exceeded. Examples of cable compaction are shown in Figure 12.34.

Figure 12.34 Cable Compaction Photos (Courtesy of Caltrans)

12.5.8.2 Oiling

The recent practice in the United States is to treat damaged cables by oiling and rewrapping them for their entire lengths. This entails major construction work and installation of work

platforms below the cables to provide access and would not normally be done by typical bridge maintenance forces. The existing wrapping is removed panel by panel, wedges are driven into the cable, and oil (usually linseed oil with or without additives) is poured into the wedged grooves. The cables are then rewrapped, usually with wire and a sealing paste and sometimes with a neoprene overwrap.

12.5.8.3 Painting

Traditionally suspension bridge cables were protected with the same paint system used for the steel structure. More recently water-based acrylic coatings that contain highly elastic polymers and cure to a rubbery coating have been used for painting suspension cables. Because of their ability to sustain up to 200 percent elongation without cracking or peeling, they have been successfully used for the maintenance painting of wire wrapped cables on many existing suspension bridges and new bridges. In addition, these coatings have proved to have a long life in other applications, especially in environments where superior salt water and chemical resistance are required. One of such materials is the proprietary coating Noxyde®, manufactured originally in Belgium and now licensed for manufacture in other countries.

It is important for bridge maintenance workers to recognize that these coatings are very different from the typical coatings used on steel bridges, and therefore should not be touched-up or spot painted without first consulting a specialist.

12.5.8.4 Cable Vibrations

A number of different methods have been considered and used on bridges to control aerodynamic instability problems in cable stays, and these include:

- Use of neoprene cushioning pads between cables and the ends of the anchor pipes.
- Cross-ties or cable restrainers have been used primarily as a temporary solution to cable stay vibration problems.
- Hydraulic dampers can serve as vibration control as permanent mitigation of rain vibration of the stay cables. The dampers act similar to automotive shock absorbers, absorbing the vibratory motion of the stay cables (motion is absorbed as strain energy), and providing additional damping to prevent further motion of the stay cable.

When to Call the Engineer

Call the Engineer when any vibration greater than 1 inch is observed.

12.5.8.5 Dehumidification

Many of the main components of major bridges are steel structures. To ensure a long service life and provide an appropriate level of safety, these structures must be protected from corrosion. Corrosion protection has traditionally been provided by means of surface treatment, such as blasting and painting. More recently, dehumidification has been developed as an

alternative method. Dehumidification has economic and environmental advantages to traditional surface treatment methods.

A dehumidification system is considered a state-of-the-art method for protecting main cables from corrosion. It is composed of a sealing system, a dry air system and a monitoring system. This is the only system which completely prevents corrosion, whereas other systems at best can only slow it down. The dehumidification system produces dry air and blows it through sections of the main cables. The system assures overpressure inside the sealed cable system. While the sealing system may have minor imperfections in the form of small leaks, no water or moisture will enter the cables (due to the overpressure).

Dehumidification systems have also been installed in the anchorage housings of most suspension bridges. As a bridge maintenance worker, it is important to understand the importance of the system. An operation and maintenance manual is usually supplied with these systems.

The dehumidification system is made up of the following main components:

- A sealing system for the main cables, including cable bands, saddles and other connected components.
- A dehumidification system capable of producing and blowing dry air through the main cables (see Figure 12.35).
- A control and monitoring system.

An example of a dehumidification plant injection site and exhaust are shown in Figure 12.36

Figure 12.35 Dehumidification Plant (Courtesy of COWI A/S)

.

Figure 12.36 Dehumidification Plant Injection Site (left) and Exhaust (right) (Courtesy of COWI A/S)

12.6 Earthquake Restrainers

12.6.1 Introduction

Bridges in seismically active regions may have earthquake restrainers attached to the superstructure as part of a seismic retrofit strategy. The earthquake restrainers may be steel rods or cables and may be visible from the ground or they may be hidden within the structure. Earthquake restrainers may be placed along the length of the bridge, vertically or transverse to the length. Regardless of the orientation, the functions of the restrainers are to hold the superstructure together during a seismic event and to prevent the superstructure from falling off its support. The maintenance of earthquake restrainers from corrosion or deterioration is similar to other steel superstructure components. Earthquake restrainers have additional unique maintenance needs that should also be considered. To effectively maintain earthquake restrainers, it is important to understand how they are intended to function during daily or seasonal thermal expansion cycles and during a seismic event, and what to do once they have been engaged during an earthquake.

12.6.2 Earthquake Restrainer Design Background

In past earthquakes, bridges have collapsed because the superstructures were displaced by seismic forces beyond the ends of the bearing supports at columns, piers, and abutments. Earthquake restrainers are rods or cables attached to the superstructure that are designed to prevent the superstructure from moving beyond their supports. Example photos and schematic designs are presented in Figure 12.37 through Figure 12.41.

Earthquake restrainers are designed to limit superstructure displacements during a seismic event; however they must accommodate normal thermal movements that occur every day. The accommodation of daily or seasonal thermal movements requires that the earthquake restrainer rods or cables be free to expand and contract without engaging the superstructure. A maintenance crew should never tighten any earthquake restrainer that appears loose or that has a sag in the cable before consulting an engineer.

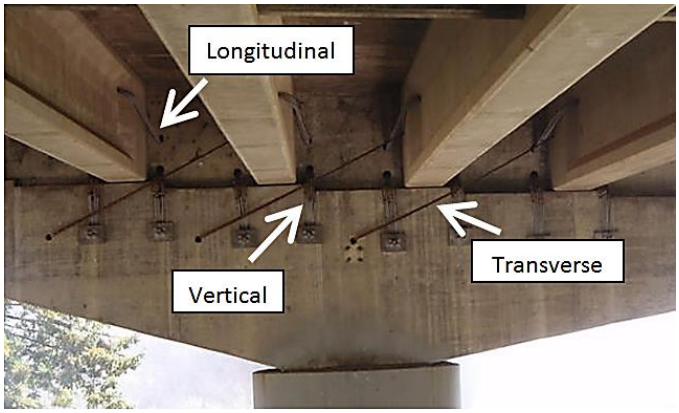

Figure 12.37 Vertical, Transverse and Longitudinal Restrainers

Figure 12.38 Earthquake Restrainer System

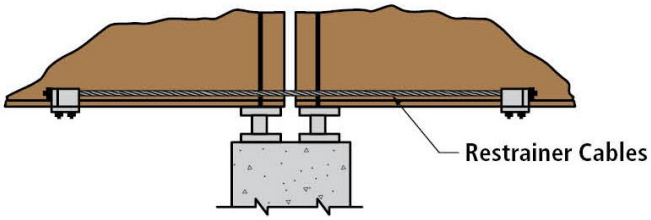

Figure 12.39 Longitudinal Restrainer of Steel Girders

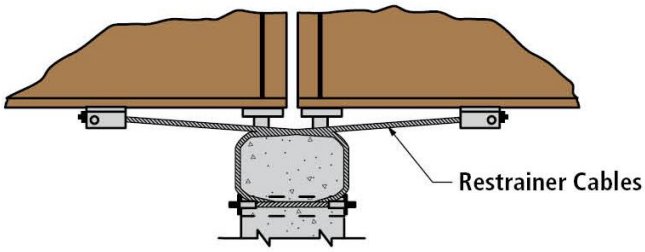

Figure 12.40 Longitudinal Restrainer of Steel Girders with Connection to Substructure

In some applications, the earthquake restrainers are designed to hold the superstructure down from seismic uplift forces. The vertical orientation is common at supports with rocker bearings or similar bearings that could topple over if the superstructure was lifted off the bearing by

seismic forces (See Figure 12.41). The vertical orientation of earthquake restrainers are typically much tighter than longitudinal or transverse applications because they need to engage quickly to restrain uplift and because their vertical orientation allows for thermal expansion without slack in the cable.

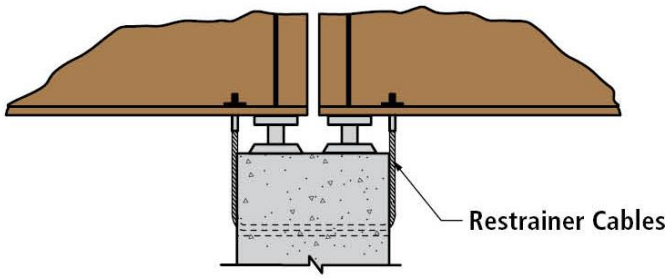

Figure 12.41 Vertical Restrainer

12.6.3 Common Maintenance Activities

12.6.3.1 Corrosion Mitigation

Earthquake restrainers are typically constructed of galvanized steel rods or cables and are fairly resistant to corrosion but not immune. Restrainers are typically around joints and can be subject to significant chloride levels from decks that are salted as the salted water drains through the joints. Coastal bridges have also been seen to have earthquake restrainers corrode to the point of replacement within 10 years of installation. Like other galvanized steel, the restrainers can be painted to protect exposed portion of the restrainers. Experience in California has shown that the cable or rod length spanning the joint is often the critical corrosion location. This location cannot be painted effectively and is also subject to abrasive wear from seasonal and daily thermal cycles that tend to wear away any surface corrosion protection. Proper joint maintenance is the key preventive measure that can be performed by maintenance crews. Flushing with fresh water can also reduce chloride levels on the steel and slow down the corrosion. Earthquake restrainers are typically fairly easy to remove and replace once corrosion begins to reduce the steel section.

12.6.3.2 Securing Restrainer Nuts

Earthquake restrainers come in varying lengths but always have threaded rods at the ends to allow nuts to be tightened to hold the restrainer in place at the ends. To allow for normal thermal expansion and contraction without engaging the restrainers, the restrainers are not tightened. Because the restrainer nuts are not torqued down, they can work their way loose from normal traffic vibrations. To prevent the nuts from coming loose, a second “jam nut” is typically placed behind the restrainer stud nut (see Figure 12.42). The jam nut is tightened against the back of the stud nut by applying torque in both nuts in opposing directions to lock in the nuts without over tightening the restrainer rod or cable. During post-earthquake inspections in California, bridges have been discovered with both nuts laying inside box girders and the cables pulled through hinges intact. Peening the threads after the jam nuts are installed should avoid nuts walking-off the end of the restrainer studs. Alternatively, a commercial thread locking product can be used after installing both nuts.

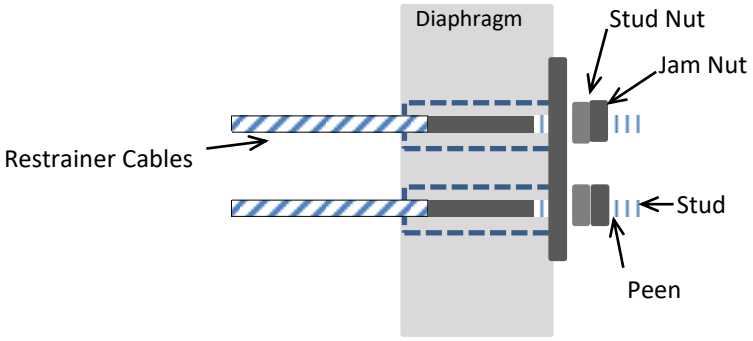

Figure 12.42 Typical Restrainer Stud End Configuration

12.6.3.3 Adjusting Cable Sag

Restrainers are designed to allow movement before they become taut in an earthquake. A sag in a cable or a loose restrainer rod is normal and should not be tightened by maintenance crews without consulting an engineer. In some cases, restrainers have lost the appropriate sag in the cables and will require adjustment. The change in cable sag can be caused by rotation of the superstructure particularly on high skew bridges or by movement of substructure elements. The need for adjustment can be identified by observing the relative sag between cables or by observed differences in tension at a single location. All cables or rods at a single location should have a similar amount of allowed displacement before engagement. If this slack distance is too little, the restrainers may engage during thermal movement possibly causing significant restrainer or superstructure damage. If the restrainer has too much slack before engagement, the superstructure may displace past the bearing causing collapse during a seismic event. To get consistent tension in the cables or rods, they can be tightened to a similar tension and then backed off enough to permit thermal movement. Alternatively, a string line can be used to measure the sag in the middle of the cable drape, and the nuts can be adjusted to achieve a specified drape distance. In either case, the maintenance crew should consult an engineer to determine the appropriate cable drape or expected thermal movement to ensure that the restrainers will function as intended during a seismic event.

12.6.3.4 Post Seismic Event Maintenance

Following a significant seismic event, maintenance crews may find earthquake restrainers that are not uniformly tensioned or broken. Restrainers are designed to work together as a system to restrain displacements during a seismic event. If the restrainers do not have uniform tension when the earthquake happens, the tightest restrainers will sustain the greatest forces and may be damaged or severed. Any restrainer that is suspected of sustaining significant seismic force (e.g., permanently elongated) should be replaced. Restrainers with any damage to the threaded stud ends should also be replaced.

Following a seismic event, it is fairly common to have damage to the concrete diaphragms that anchor restrainers. The damage can range from cracking to significant spalling. In most cases, diaphragm damage can be repaired using techniques covered within this chapter. In cases of severe diaphragm damage, the diaphragm may need to be replaced or supplemented with a second diaphragm behind the original.

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Chapter 13 - Substructures

13.1 Preventive and Basic Maintenance of Concrete Substructures

Preventive Maintenance (PM) for reinforced concrete, steel, and timber substructures is an effective method of avoiding costly repairs. PM includes removing debris and pressure-washing seats, caps and other surfaces exposed to salt, maintaining coatings systems, and avoiding rot and vermin attack.

Leaking joints are commonly evidenced by discoloration of the sides of the substructure. Eventual concrete deterioration and spalling ensues. On structures with end diaphragms that extend down to the top of the cap, the water drains down the end of each cap, thereby causing significant concrete deterioration.

Protection against salt-water damage is the same as on the deck. Coatings are effective for steel or concrete if applied early and reapplied when needed. Dense concrete, such as that attained with a pozzolan additive, is also effective in resisting salt-water damage.

When performing preventive maintenance on a substructure, this is also a good time to check if the bridge may be in distress. Guidance on items to be checked for concrete substructures follows.

What To Look For

Concrete Substructure Components

- Look for cracking and possible movement in the abutment walls. Check particularly the construction joint between the backwall and the breast wall.
- Look for scour or erosion around the substructure, or evidence of any movement.
- Look for loss of bearing area.
- Look for large cracks, greater than 1/8 inch wide.
- Look for impact damage.
- Look for efflorescence, spalling, and scaling of concrete.

Typical areas of concern are shown for concrete abutments and piers in Figure 13.1 and Figure 13.2, respectively.

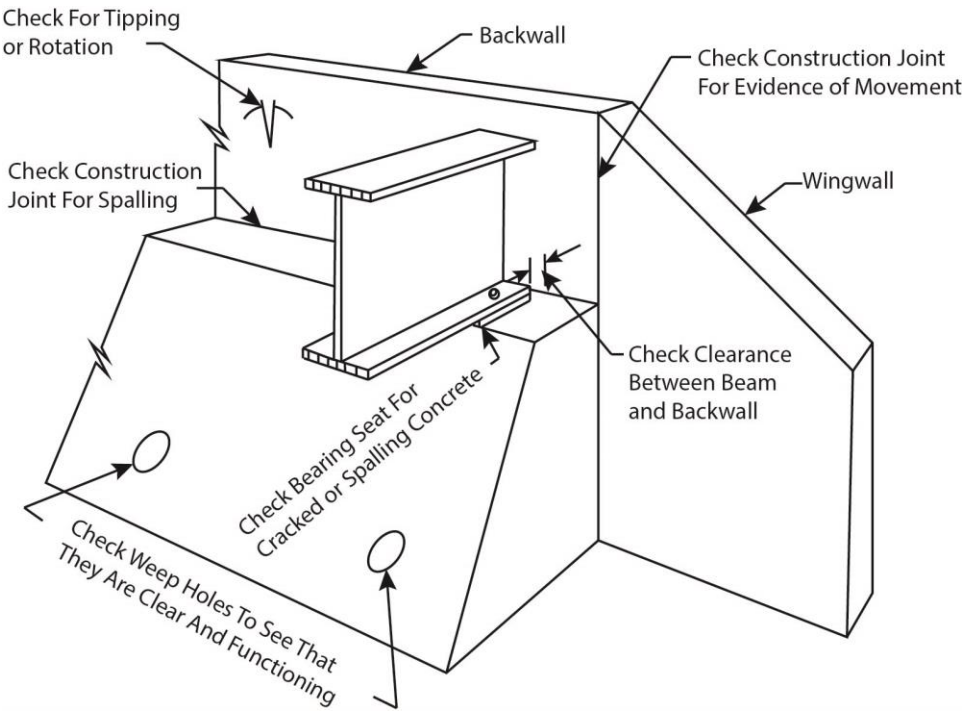


Figure 13.1 Items to Look For at Concrete Abutments

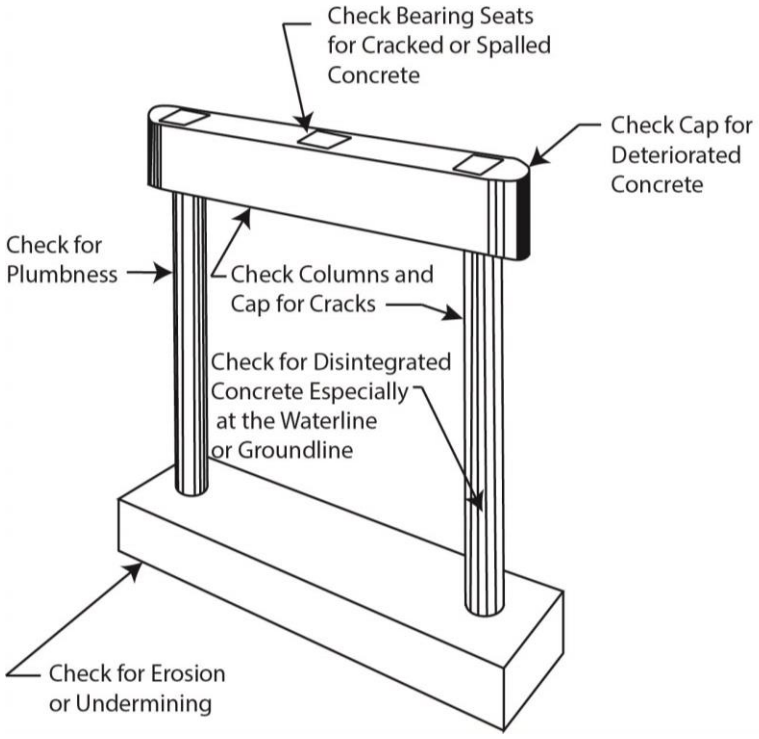


Figure 13.2 Items to Look for at Piers

13.1.1 Cleaning

Debris such as road grit, animal droppings and de-icing salts create a serious problem if not removed. Moisture, deicing salts, and debris that spill through leaking or open joints tend to accumulate and pile up on the seats and the top of the cap. Soon the by-products of

corrosion of the rebar cause the concrete to spall. Accumulated debris also hampers inspection efforts.

Birds are common inhabitants on bridges. The wide flanges of steel girders and pier caps are preferred nest and roosting sites. These areas should be thoroughly washed and installation of bird screens would be beneficial. Figure 13.3 shows bird droppings and corrosion of the bearings. Horizontal cracking of the pier cap is also evident, occurring from corrosion of the top mat of reinforcing steel. If bird or nest removal is required, caution should be exercised as there may be restrictions due to federal protection of birds or other species such as bats.

Removal of the droppings should be done while wearing a respirator to avoid inhaling contaminants. Histoplasmosis is an infection of the lungs leading to a cough or flu-like symptoms which is often associated with decaying bat guano and bird droppings. Disruption of the debris can release infectious elements that are inhaled and settle into the lung.

Figure 13.3 Debris on Pedestal and Pier Cap

The bearing seats can also be affected. When the bearings become “frozen” by corrosion, additional stress is introduced into the substructure that can cause spalling or damage to the bearing system. Overloading of the bearing system can result in fracture of the substructure cap and loss of bearing for the beam ends.

Removing the debris must be done before washing. Debris is collected and, if allowed is wasted upland away from the roadway and drainage areas. Some agencies remove debris annually. In some locations, the accumulation occurs so quickly crews remove material monthly. Respirators may be necessary when removing debris.

Debris under the bridge should also be removed. Sometimes the space under the bridge has been used for illegal dumping, drugs, and the homeless. An example is shown in Figure 13.4. Crews should be cautious when encountering unknown materials, syringes, and other hazards left by people seeking shelter within the bridge confines.

Figure 13.4 Bridges Sometimes Become Makeshift Homes

Debris or floating ice that drifts against the substructure can cause premature deterioration and place excessive lateral loads on the whole structure. The techniques available to remove drift are:

- Clear small debris with a pole or hook.
- Pull large pieces of debris clear with a crane or excavator.
- Clear large and small pieces of debris by flushing with large volumes of water.
- Ice jams can either be blasted apart (if permitted) or broken up by flushing with large volumes of water or excavator if the jam can be reached.

Additional information on debris removal can be found in Chapter 15 - Channel and Waterway.

Washing substructures can be done using a volume flush or pressure washing. Flushing requires considerable more water than pressure washing. An example of flushing is shown in Figure 13.5. Tanker trailers, fire trucks and old milk trucks are used to carry water to bridge sites.

Figure 13.5 Flushing with Water

Pressure washing, typically with 2,500 to 3,000 psi pressure uses much less water and, though time consuming, provides for a cleaner surface. In most locations, the amount of pressure that is used depends on the condition of the paint. In some cases, the pressure limitation is stated as

"a pressure that does not remove paint". Some states require pressure washing in preparation for the application of a penetrating sealer. Water tanks with baffles can be loaded onto stake racks if crews need to bring water to the site. It may be permissible to pump water from streams for wash water. Crews should check with an environmental coordinator if they are allowed to perform this type of operation.

The bridge expansion joints, drainage pipes, scuppers, downspouts, pier and abutment caps and seats should be cleaned. Access is often a concern, particularly for piers in the water. In some cases, as shown in Figure 13.5, access to the substructure may involve equipment, such as an under bridge unit or scaffolding. Bridge cleaning is typically done in the spring. This minimizes the time that salt is in contact with the bridge and occurs when streams are commonly at their highest levels of flow.

13.1.2 Graffiti Control

Graffiti removal commands a great deal of time and resources for some agencies. Graffiti removal is scheduled by highway route segments for intensive clean-up. Graffiti that was once seen only on walls is now showing up on overhead signs, retaining walls, bridges and bridge columns. With increasing cost for graffiti removal and an increasing need for abatement, new and innovative methods are being explored.

Graffiti vandalism is a crime and for this reason it is recommended that agency retain photographic evidence of incidents of graffiti vandalism. Graffiti is a community-wide problem with both public and private facilities being affected and seriously impairs community appearance and values.

It is recommended that rapid removal is the first strategy adopted in graffiti management. Rapid removal is recognized as best practice because it reduces the level of recognition sought by vandals. Rapid removal also prevents assets from looking neglected. In the case where graffiti is racist or obscene, the graffiti should be removed immediately after taking a photograph.

Urban art projects have provided positive results where a wall with a painted mural is less of a target for graffiti vandalism than a blank wall. There are urban artists who can be contracted to come into the school and take students through a workshop of urban art/mural painting. Several local councils or police stations are equipped to provide graffiti education in schools. In some cases, local high school bridge graffiti was eliminated when students painted "Welcome to Our School" or similar on the bridge.

Concertina wire can be wrapped around columns or end posts as a way of discouraging taggers from gaining access to the overhead signs on heavily targeted areas. Rat Guards, a steel cone personnel barrier, is placed on sign posts to deter taggers from scaling the poles. The guards are used at favorite tagging locations. The guards are made of smooth metal and are disk-shaped, making it difficult to climb on to the overhead signs. Cobra Shields have been used in the last few years. The shields, which vary in length and width, extend over the front and sides of the overhead signs, making it difficult for someone to reach over and deface the signs.

Anti-graffiti coatings are also available. The material is painted on clean surfaces and creates a non-stick surface that repels graffiti from paint, paint spray cans and permanent markers. Graffiti is removed by pressure washing or hand wiping.

13.1.3 Abutment Weepholes

Weepholes are built into abutments to allow water that collects from behind the abutment to drain. The weepholes should be cleaned out and inspected to be sure they are functioning. If water is allowed to collect behind the abutment, it may lead to tilting, settlement, or create problems with the foundations, due to the hydrostatic pressure of water behind the abutment wall placing an additional load on the wall that it was not designed to withstand. Installing rodent guards is often not a bad idea, if there is evidence of rodent habitation. Figure 13.6 shows initial abutment construction with weepholes in place.

Figure 13.6 Weepholes Consisting of PVC Pipe

13.1.4 Sealers

Concrete sealers are used to prevent chloride ions from diffusing into the concrete. Measures that prevent water from entering the concrete will also minimize chloride intrusion. However, chloride ions that already exist in the concrete, especially near the surface, will diffuse into the concrete after the sealer or coating is applied and may critically contaminate the concrete at the reinforcement level over time. Workers should be reminded that sealants and coatings contain solvents that will flash off during installation and some can be harmful in confined spaces.

There are generally two categories of sealers: penetrating sealers and surface sealers. Sealers prevent liquid water from entering the concrete, but they are generally "breathable". This means that they allow water vapor to enter and leave the concrete. This prevents moisture from being trapped inside when the concrete is sealed, thus minimizing the possibility for freeze/thaw damage of the concrete, corrosion damage, or reactive aggregate damage. This is especially important when the concrete member is in contact with earth and there is ready access of moisture into the concrete.

Vapor permeability of concrete sealers is desirable because it prevents moisture from being trapped inside the concrete element when the concrete is sealed. Acceptable sealers should be able to provide:

- Chloride screening: Sealers should be able to reduce chloride ingress into concrete by at least 90 percent after 30 weeks of ponding with saltwater.
- Penetration depth: The initial depth of penetration should be 0.125 inch, and ideally 0.25 inch, to provide for protection from wear and UV light degradation.

- Moisture vapor permeability: The minimum vapor transmission should be 80 percent after sealing of the concrete. The percentage of vapor transmission is determined by comparing the vapor loss of sealed concrete to that of an unsealed concrete over a 14-day period. The concrete used in the test should be in a saturated, surface-dry condition.
- Surface friction: The surface should exhibit acceptable frictional characteristics after it is sealed

Penetrating Sealers

Penetrating sealers prevent liquid water from entering the concrete; however, they are very permeable to water vapor. Silanes and siloxanes have proven to be the most effective sealers in laboratory testing. As substructure concrete is not abraded by traffic, the time between resealing is roughly twice that of concrete decks. In addition, as dry time is not critical a wider variety of sealers can be applied to substructures.

Water based or 100 percent solids products should be used when sealing substructures near waterways. Non-water based products or those not containing 100 percent solids contain volatile organics that are harmful to aquatic life and can contaminate drinking water. It is best to use water-based products near waterways.

A sealer will not penetrate properly, unless it is applied to a clean surface. Before application, any surface laitance or residual curing compounds should be removed from the concrete surface through pressure washing, sandblasting, or shotblasting. Surfaces are cleaned using pressure washing with 2,500 to 3,000 psi after removing debris. Oil, asphalt, and other surface contaminates that would interfere with the penetration of the sealer should be removed during the cleaning process. Following cleaning, any dust or loose matter should be removed with compressed air or vacuum.

To provide for proper penetration, the subsurface pores must be dry to the desired depth of penetration. Newly placed concrete must be allowed to cure for a minimum of 28 days (or longer if recommended by the manufacturer) before sealing. If after curing concrete is exposed to rain, it must be allowed to dry. The drying requires a sufficient period of dry, warm weather as outlined in Table 13.1.

Table 13.1 Required Drying Time before Sealing (Days)

Ambient Temperature	Last Rain - Light	Last Rain - Moderate	Last Rain - Heavy
70 - 85 °F	0.5	1	2
50 - 70 °F	1	2	3
40 - 50 °F	2	3	5

Penetrating sealers, silanes and siloxanes, are typically applied to a prepared concrete surface at a rate of about 150 square foot per gallon. Sealers should not be applied when the temperature of the concrete surface is below 40 °F. The sealer should be applied from the low area to the high area to provide for proper saturation. Sealers may be applied by low-pressure

pump (with either nozzle or spray bar) and by flood and brush techniques. Substructures can be sealed using simple equipment such as the garden sprayer on a backpack as seen in Figure 13.7.

Figure 13.7 Hand Sealing the Substructure

Surface Sealers

Surface sealers are pore-blocking materials such as boiled linseed oil (50 percent linseed oil and 50 percent mineral spirits or kerosene) or epoxy with low solid content (solids less than 50 percent). Surface sealers have an inadequate depth of penetration and quickly wear when exposed to abrasion.

Environmental exposure conditions that influence the service life of sealers applied to substructure components include UV light, moisture, and abrasive wave and ice action. Surface sealers not exposed to abrasive wave or ice action have a service life of 1 to 3 years. In the presence of abrasive wave or ice action, the service life of surface sealers may be less than 1 year. This is not a very long service life for concrete preservation, so the use of coatings as discussed in Section 13.1.5 is typically recommended.

Some states apply the protective coating to the bearing systems as well as to the substructure caps. When coating concrete caps, the best procedure is to extend the protective film on the abutments to a minimum of one foot below the bridge seat. On piers, the extent of the coating may depend, to some extent, on appearance.

13.1.5 Coatings

A coating is a one- or two-component organic liquid containing a polymer binder (such as epoxies, acrylics, methacrylates, or urethanes) applied in one or more coats to a prepared concrete surface. The primary purpose of the coating is to prevent the ingress of water into the concrete and, hence, the diffusion of chloride ions. Coatings prevent liquid water from entering the concrete, but they usually do not permit water vapor transmission. Therefore, concrete members should be dry if a coating is applied. Also, the member should not be in contact with earth with ready access of moisture into the concrete or bubbling, flaking, failure of the coating may result.

The selection of a coating material depends on individual site conditions. Polymer surface treatments cure quickly. Epoxies are abrasion resistant and have a high adhesive strength; however, they are susceptible to degradation by UV light. Acrylics are brittle and normally have

low impact strength. Urethanes have high impact strength and good weathering characteristics, but low abrasion resistance. Coating materials have high solid content, usually 100 percent, and the typical thickness of coatings after drying is in the range of 0.001 to 0.003 inch.

Coatings applied to existing concrete may prevent further chloride intrusion, but if chloride ions are already present in the concrete, they may still critically contaminate the concrete at the reinforcement.

The performance of any coating system is contingent upon proper surface preparation. The surface should be sandblasted to remove laitance. Before coating, the surface should be thoroughly air-blasted to remove dust and debris. The subsurface pores must be dry to minimize vapor pressure build-up on coatings. If concrete is exposed to rain, it must be allowed to dry. The drying requires a sufficient period of dry, warm weather.

Coatings should be able to reduce chloride penetration into the concrete by at least 90 percent and have a minimum vapor transmission of 35 percent.

Coatings should be mixed prior to placement to ensure a uniform mix. Once the coating components have been allowed to react, the coating may be applied by brush, roller, or spray as necessary to ensure even coverage, as shown in Figure 13.8. The coating should be applied in accordance with manufacturer's recommendations which are typically between 50 °F and 90 °F. Two applications of the coating should be applied to ensure even coverage. The second coat is usually applied 24 hours after the application of the first coat, but this can vary with environmental conditions and material type.

Figure 13.8 Protective Coating Applied on the Substructure

If the coating is applied on horizontal surfaces, sand should be broadcast on the coating after each application and while the coating is still wet. This is to impregnate the coating and provide skid resistance on walking surfaces. Before coating, all bearings should be masked off. This is done to prevent accidental application on the bearing, which may affect its performance. The coated surface should be protected from rain and traffic spray for at least 6 hours after application.

The service life of coatings depends on the type of coating material applied and the field exposure conditions. Coated bridge components subjected to sea spray may have a shorter life than those exposed to deicer salt runoff water. For coatings on substructure components,

environmental exposure conditions that influence the service life include degradation caused by ultraviolet light, moisture, and abrasive wave and ice action. Depending on the type of coating, the service life varies from 6 to 14 years for coatings subject to sea spray and 10 to 18 years for coatings subject to deicer salt runoff water. This is substantially higher than surface and penetrating sealers discussed in the prior section.

Cementitious and epoxy coatings have been applied under water to protect a concrete surface against abrasion, cover cracks, and make small repairs. These products are normally applied by hand as a thick mortar. The cementitious products often include anti-washout admixtures and accelerators. Cementitious materials are mixed above water and delivered to divers in a plastic bag. Epoxy resins applied under water must perform satisfactorily and cure under wet and low surface temperature conditions.

13.1.6 Cathodic Protection

Cathodic protection (CP) was first used on bridge decks in the 1970's and on bridge substructures in the 1980's. Since then, it has been employed by a number of state Departments of Transportation on bridges that experienced reinforcing steel corrosion due to deicing salts or marine exposure, or both. While earlier cathodic protection installations on bridges were of the impressed current type, significant advances have been made in the past decade in adapting galvanic systems, which are simpler and more easily maintained, for substructures in particular. Both will be discussed in this section. CP results have been mixed, with some states employing cathodic protection extensively and others only on a limited basis or not at all. The widely differing levels of usage may be due to such factors as:

- Initial and ongoing costs
- The technology, including design, installation, and maintenance, is not understood by many highway personnel
- Instances of inadequate performance
- The absence of a protocol for deciding when to use cathodic protection, considering factors such as type of structural element, nature of exposure, extent of corrosion-induced damage in concrete, desired service life, etc.
- The wide array and evolution of materials and instrumentation available in the marketplace

The Federal Highway Administration has stated that cathodic protection is the only rehabilitation technique that has proven to stop corrosion in salt-contaminated bridge decks regardless of the chloride content in concrete. A more recent evaluation of various types of cathodic protection systems suggests that cathodic protection is effective if designed, installed, and operated in accordance with recommended practice. Although cathodic protection can be applied during new construction, it is most often found in conjunction with rehabilitation of existing concrete structures. However, cathodic protection is not always needed, nor is it applicable on every structure.

The design of the specific cathodic protection system depends upon the type of surface to be protected (horizontal, vertical, soffit, etc.), geometry, reinforcement cover depth, environmental considerations (temperature and moisture), and structural considerations (additional dead load capacity). Titanium-anode-based cathodic protection systems can be

expected to last over 50 years under certain circumstances. For this reason specialty firms are usually brought in to perform the design.

Cathodic Protection is a technique used to control the corrosion of a metal surface by making it the cathode of an electrochemical cell. A simple method of protection connects protected metal to a more easily corroded "sacrificial metal" to act as the anode. The sacrificial metal then corrodes instead of the protected metal.

A cathodic protection system works much like a battery. Batteries have a cathode and anode as show in Figure 13.9. Similarly, a bridge can be turned into a giant battery with the goal of protecting the steel reinforcement.

Figure 13.9 Cathodic Protection requires a Cathode and Anode

When a concrete bridge component is severely contaminated with deicing salts, cathodic protection (CP) may be used over conventional treatments to stop and/or prevent the corrosion process in concrete. This is because, unlike conventional treatments, CP does not require removal and replacement of sound, but nonetheless, contaminated concrete in order to stop and/or prevent the corrosion. Compared with the cost of concrete removal and replacement, CP may be less expensive. The supplier of the system will supply specifications and instructions for installation and maintenance. Substructure units can be protected by cathodic protection.

Galvanic CP System

This system is based on the principle of dissimilar metal corrosion and the relative position of specific metals in the galvanic series. No external power source is needed with this type of system and much less maintenance is required. The anode sacrifices itself to protect the reinforcing steel from corrosion. Over time the anode will be exhausted and will no longer provide protection. The basic layout is shown in Figure 13.10.

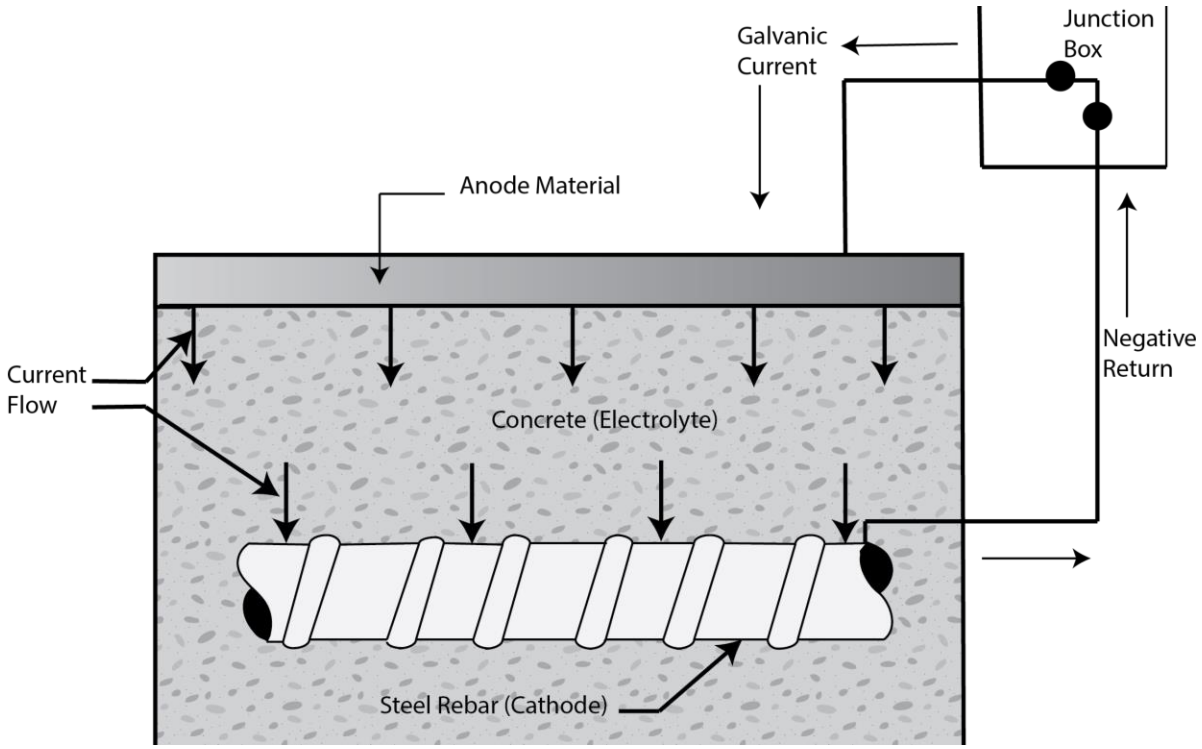


Figure 13.10 Layout of Galvanic (Sacrificial) Anode System of the Reinforcing Steel

Thereby, corrosion stops, or at least is greatly minimized. CP is divided into two basic types: 1) galvanic (or sacrificial anode systems, and 2) impressed current systems. In both types of systems, the reinforcing steel functions as a cathode, hence the name cathodic protection.

Such system provides protective current primarily to the areas on the steel surface that need it the most. Due to the limited power provided by this system, most of the installations have been on bridge components in marine environments or areas with high humidity where the concrete resistivity is generally much lower. Recently, galvanic systems are gaining more widespread usage, particularly for substructure applications.

Galvanic (sacrificial anode) Systems are made from a metal alloy with a more "active" voltage (more negative electrochemical potential) than the metal of the structure. The difference in potential between the two metals means that the galvanic anode corrodes, so that the anode material is consumed in preference to the structure. The loss (or sacrifice) of the anode material gives rise to the alternative name of sacrificial anode.

For this to work, there must be an electron pathway between the anode and the metal to be protected (e.g., a wire or direct contact) and an ion pathway between both the oxidizing agent (e.g., water or moist soil) and the anode, and the oxidizing agent and the metal to be protected, thus forming a closed circuit; therefore simply bolting a piece of active metal such as zinc to a less active metal, such as mild steel, in air (a poor conductor and therefore no closed circuit) will not furnish any protection.

There are three main metals used as galvanic anodes including magnesium, aluminum and zinc. They are all available as blocks, rods, plates or extruded ribbon. Magnesium has the most negative electro-potential of the three and is more suitable for areas where the electrolyte (soil

or water) resistivity is higher. This is usually for buried portions of substructures. Zinc and aluminum are generally used in salt water, where the resistivity is generally lower.

As the anode materials used are generally more costly than iron, using this method to protect ferrous metal structures may not appear to be particularly cost effective. However, consideration should also be given to the costs incurred by closing a bridge to repair a pier column in the water or the need to shore a pier cap because of their structural integrity has been compromised by corrosion.

There is a limit to the cost effectiveness of a galvanic system. On larger structures, so many anodes may be needed that it would be more cost-effective to install impressed current cathodic protection.

Substructure anode systems are of two types: 1) encapsulated or embedded and 2) surface applied. The encapsulated or embedded systems use an anode, such as zinc anchored to concrete (e.g., the pier surface) and encapsulated in concrete (e.g., formed jacket or shotcrete). A schematic illustration of a zinc mesh jacket is presented in Figure 13.11. It is a two piece snap together system.

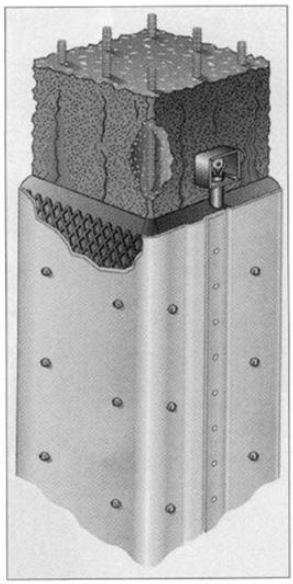


Figure 13.11 Schematic Illustration of a Zinc Mesh Jacket

The two most common galvanic anode systems for substructures in a marine environment are 1) Thermally Sprayed Zinc (TSZ) and 2) Zinc-hydrogel anode. The application of thermally sprayed zinc is the same as used in the impressed current system. Except in the case of the galvanic system, the zinc may be applied directly to cleaned steel in areas where damaged concrete has been removed and to the adjacent concrete surfaces. If structurally and aesthetically acceptable, the areas where damaged concrete was removed may be left unrepaired.

Thermally sprayed zinc for cast-in-place substructure components and zinc mesh jackets for precast ones are common means to control this corrosion and extend useful service life. For both types of systems, a submerged bulk zinc anode (SBA) is included to polarize the reinforcement below the waterline and reduce current drain from the lower portion of the thermal spray or zinc mesh. See Figure 13.12 for a schematic illustration of thermal spray Zn

with a Submerged Bulk Anode (SBA) system applied to a pier and Figure 13.13 for an example of thermally sprayed zinc applied to a pier column.

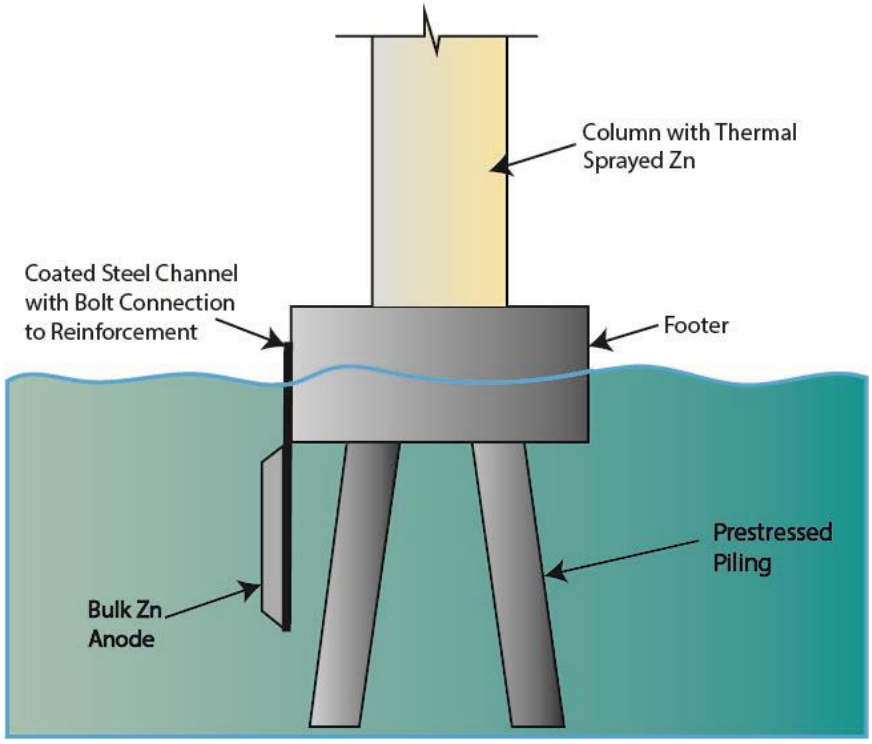


Figure 13.12 Schematic Illustration of Thermal Spray Zn/SBA System of a Substructure

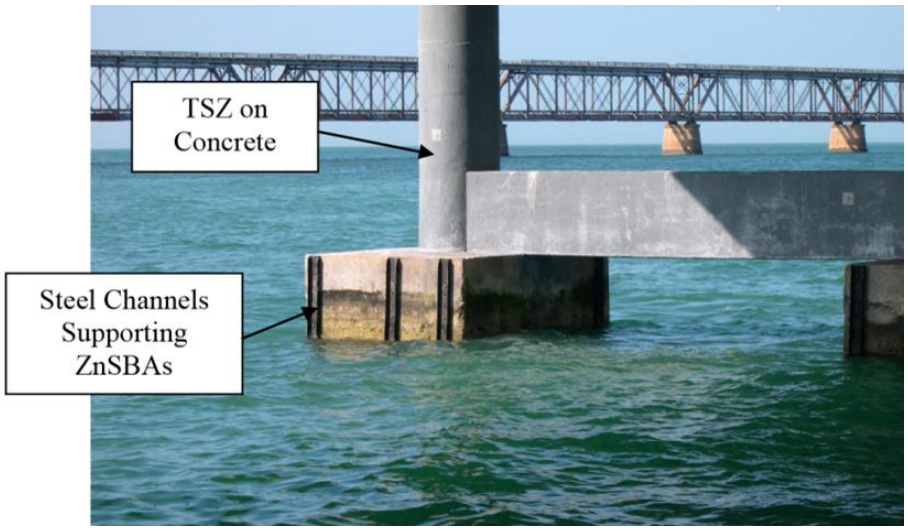


Figure 13.13 Thermally Sprayed Zinc Applied to a Pier Column

The zinc-hydrogel anode system is also a sacrificial anode systems. A very thin sheet of zinc is glued to the prepared concrete surface by means of a hydrogel glue (basically the same as the material used for electrode attachment to skin in medical applications). It should be noted that the gel is a hydrophilic material and attracts moisture. Therefore, this system should not be used in areas with potential to water exposure. Otherwise, moisture will accumulate at the interface of the gel and zinc and can cause deterioration in time. The zinc sheet can be coated

for aesthetics, if desired. This system is generally more effective than the thermally sprayed zinc system.

Impressed Current System

An impressed-current cathodic protection is achieved by driving a low-voltage direct current (generally less than 50 Volts) from a relatively inert anode material, through the concrete, to the reinforcing bars. Direct current of sufficient magnitude and direction is applied to oppose the natural flow of current resulting from the electrochemical process. The direct current is supplied by an external power source, often a rectifier that converts alternating current to direct current. Recently, solar power and specially designed batteries have been successfully used as external power sources. The direct current is distributed to the reinforcing bars by an anodic material. The uniformity of the current distribution is critical; therefore, an anode is one of the most important components of a cathodic protection system. Current distribution is also a major consideration in the design of cathodic protection systems. A basic layout required for impressed current CP systems is presented in Figure 13.14. Impressed current systems should be monitored periodically to ensure that correct amount of voltage and amperage is applied to the reinforcing steel. Monitoring is done through reference half-cells embedded in concrete in the vicinity of reinforcing steel.

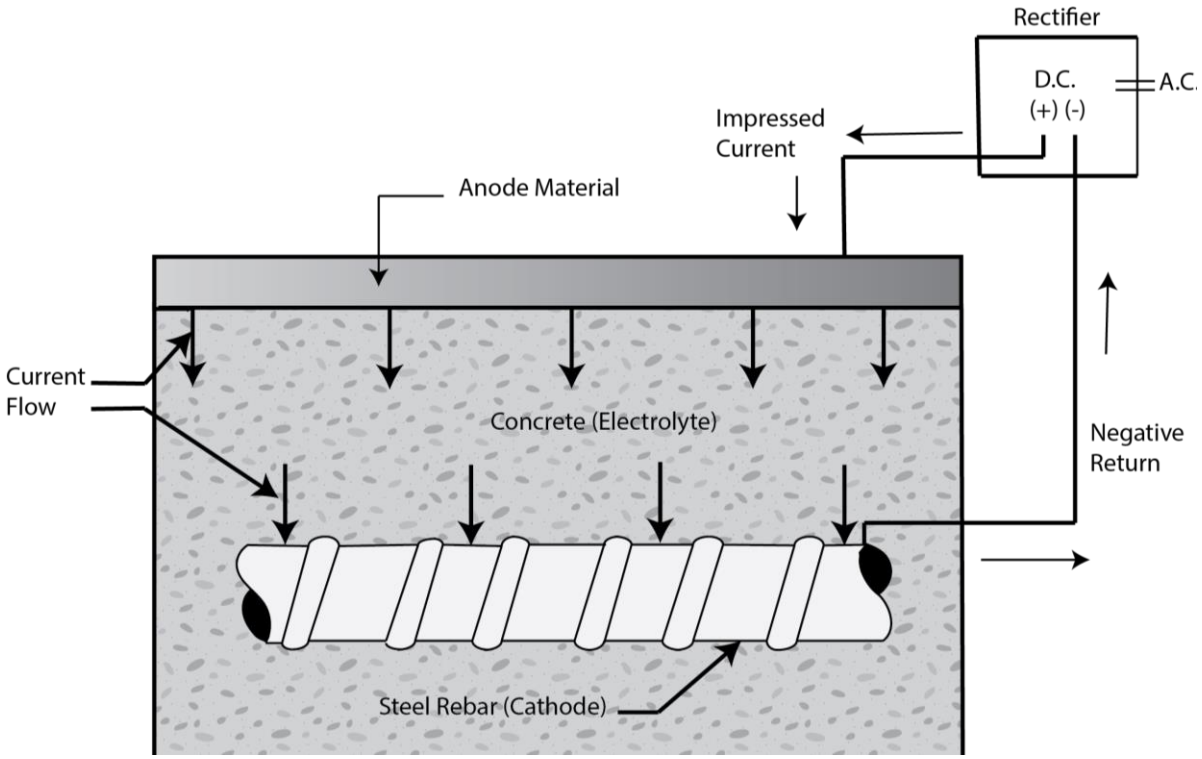


Figure 13.14 Basic Layout of Impressed Current System

Anodes for impressed current systems are available in a variety of shapes and sizes. Common anodes are tubular and solid rod shapes or continuous ribbons of various materials. These include high silicon cast iron, graphite, mixed metal oxide, platinum and niobium coated wire and other materials.

13.1.7 Electrochemical Chloride Extraction

Electrochemical chloride extraction is similar in principle to cathodic protection. However, the total amount of charge (i.e., current with time) is approximately 50 to 500 times that used for cathodic protection. Another important difference is that chloride extraction is a short-term treatment, whereas cathodic protection is normally intended to remain in operation for the life of the structure.

Chloride extraction is particularly suited for structures in which active corrosion is occurring, but only minor concrete damage is present. In addition, the structure must be conventionally reinforced and have an expected remaining service life of at least 5 to 10 years. Chloride extraction is accomplished by applying an anode and electrolyte to the structure's surface and passing direct current between the anode and the reinforcing bars, where the reinforcement acts as a cathode (as shown in the schematic in Figure 13.15). Conduction of direct current through the concrete is accomplished by the movement of charged ions. Since anions (i.e., negatively charged ions) migrate toward the positively charge anode, it is possible to cause chloride ions to migrate away from the reinforcing bars. The speed at which this process is accomplished largely depends on the magnitude of the applied current.

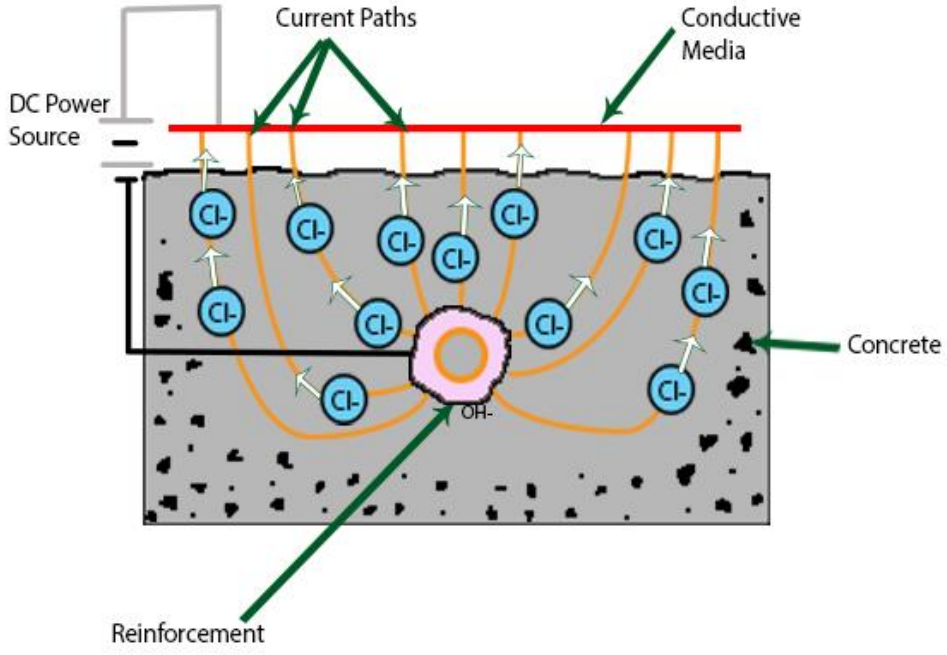


Figure 13.15 Schematic of Chloride Ion Movement in an Electrochemical Extraction

13.2 Preventive and Basic Maintenance of Steel Substructures

Under ideal conditions, structural steel will last indefinitely with no strength loss. However, when steel is exposed to electrolytes, such as water or soil, it becomes a conductor of current. If there are dissimilar conditions in the metallic conduction circuit or in the electrolyte, a corrosion cell is formed. The rate of corrosion is determined by the magnitude of the current flow and this is controlled by the degree of dissimilarity between the conditions.

The main methods of corrosion prevention are:

- Good housekeeping
- Keep drains open
- Keep exposed areas clean (flush or pressure wash)
- Spot paint as necessary
- Painting
- Galvanizing
- Cathodic Protection

Corrosion of structural steel is usually easy to locate by visual inspection and can generally be controlled with proper preventive maintenance. Items which may indicate corrosion of steel substructure components are shown below.

What To Look For

Steel Substructure Components

- Look for any cracked or broken members
- Look for bent, bowed or distorted steel element
- Where rocker bents are designed to rotate freely on pins and bearings, look to see that such movement is not restrained.
- Look to see if steel pier caps have rotated
- Look for section loss greater than 20 percent

Preventive maintenance of steel bridge substructure components consists mainly of measures to protect the steel from corrosion. The preservation of steel involves protection from exposure to electrolytes, such as water or soil. When deicing salt is added to the electrolyte, there is a dramatic increase in the rate of corrosion of the structural steel. Corrosion is usually easily spotted by visual inspection.

13.2.1 Protection of Steel

Protection from corrosion can take various forms, including:

Weathering steel: This special type of steel forms its own protective coating and theoretically does not need painting. However, many state highway departments have indicated variable performance from bridges constructed with this type of steel. The protective patina is formed as the steel goes through wet/dry cycles. In locations that remain moist, either from debris, high humidity (i.e., near a waterfall), or persistently in the shade, the patina does not form and the steel continues to rust. Sheets of very thin, flake rust can easily be removed by hand or hammer, eventually leading to significant section loss and loss of strength. Substructures constructed from weathering steel should be monitored for excessive corrosion and painted if necessary.

Paint: Typical painting requirements are based on whether the steel is new or is to be repainted. Bridge paint is often a three coat system, with a prime coat (commonly a zinc-rich primer), an intermediate coat (for moisture protection), and a top coat (for weathering

resistance). It is critical to follow manufacturer's recommendations to maximize the longevity of the paint system. Coatings are covered extensively in Chapter 18.

Cathodic Protection: Zinc or aluminum anodes are attached to H piles to abate corrosion of steel in salt or brackish water. Small zinc anodes are used when less than 8 linear feet of pile is exposed. Large zinc or aluminum anodes are used when greater than 8 linear feet of the pile is exposed.

Hot-dip galvanizing is recommended for new, replacement, or existing components. Zinc-rich paint may also be used. Care should be taken to touch up the protective coating after tightening the nuts on anchor bolts and other rail connections.

Galvanizing generally refers to hot-dip galvanizing which is a way of coating steel with a layer of metallic zinc. Galvanized coatings are quite durable in most environments because they combine the barrier properties of a coating with some of the benefits of cathodic protection. If the zinc coating is scratched or otherwise locally damaged and steel is exposed, the surrounding areas of zinc coating form a galvanic cell with the exposed steel and protect it from corrosion.

Galvanizing, while using the electrochemical principle of cathodic protection, is not actually cathodic protection. Cathodic protection requires the anode to be separate from the metal surface to be protected, with an ionic connection through the electrolyte and an electron connection through a connecting cable, bolt or similar. This means that any area of the protected structure within the electrolyte can be protected, whereas in the case of galvanizing, only areas very close to the zinc are protected. Hence, a larger area of bare steel would only be protected around the edges.

13.2.2 Cleaning

Pressure spray or hand cleaning is satisfactory for removing dirt and debris. Good housekeeping is the most effective method of protecting steel from corrosion.

Metal Substructure Cleaning:

- Keep drains open to remove standing water from steel surfaces
- Keep deck joints watertight to prevent water leakage onto steel members
- Keep exposed areas clean by pressure washing
- Spot paint and repaint as necessary

13.2.3 Blast Cleaning

Corrosion or deterioration in steel piling usually occurs at the water or ground line where wet and dry conditions alternate. Damage from impact may also be incurred where piles are near roadways or waterways.

Rust and corrosion should be removed by sandblasting or pneumatic needle scalers. Damaged areas should be straightened when possible (see Section 11.5.4) and the deteriorated areas strengthened by welding or bolting steel plates extending far enough above and below the deteriorated area to restore the full load carrying capacity of the pile. Where deterioration is of a minor nature, plating by adding metal welds may be used. All repaired areas should be cleaned and painted as good preventive maintenance.

13.3 Preventive and Basic Maintenance of Timber Substructures

Preventive maintenance of timber substructures usually involves spotting decay early and taking action. Identifying areas of high moisture content and then removing or diverting the moisture source is critical in protecting a timber bridge from premature deterioration.

A trained bridge maintenance worker who is equipped with a flashlight and probe can accomplish much. Often the greatest challenge is having the time to get to sections of the bridge where decay is most likely. For an example of common timber inspection items, please see Figure 13.16. Pile caps are subject to deterioration if water from the bridge deck is able to seep into checks on the top face of the piling. Decay of wood pilings is most likely to occur immediately below the groundline.

Timber abutments are typically pile bents acting as soldier piles with wales and lagging to retain the fill.

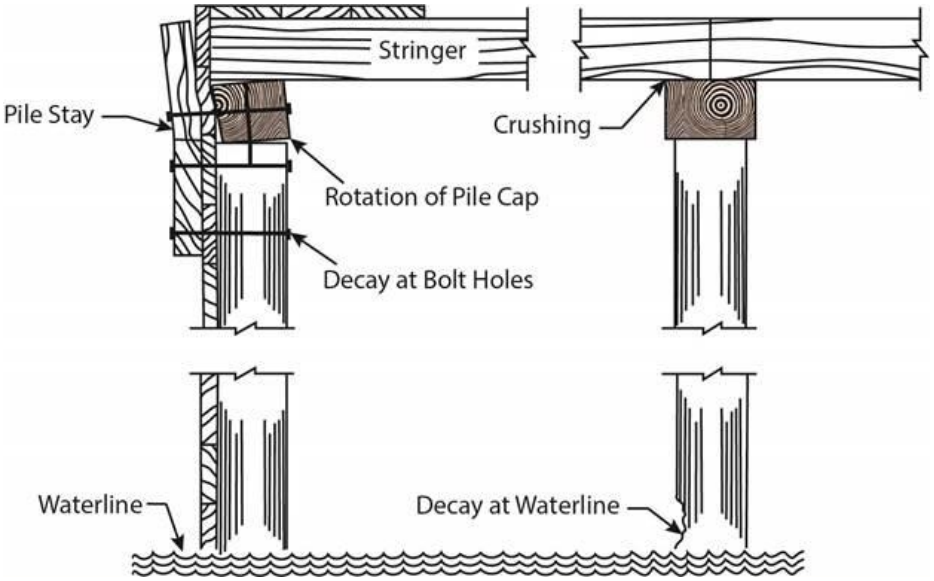


Figure 13.16 Common Timber Inspection Items

Bents are intermediate supports between abutments for multi-span crossings. They consist of timber piles or frames, depending on their height and foundation material. Pile bents are practical if foundation material is suitable and the bent height plus pile penetration is within an acceptable limit. Framed bents may be supported on footings or piles, depending on the quality of the foundation. See Figure 13.17 for a schematic of common timber bent inspection items.

Items to look for in timber substructure components are presented below.

What To Look For

Timber Substructure Components

- Check for decay in the piles, caps, and bracing. Decay may be found by tapping with a hammer or by test boring the timber. In simple terms, active decay makes timber look like mulch.
- Untreated timber will usually rot from the surface in and is normally visible.
- Check particularly at the groundline, waterline, and at joints and splices, since decay usually begins in these areas.
- Check hardware at splices and connections for tightness and corrosion. The acidity associated with corrosion causes cellulose hydrolysis, which severely reduces wood strength. Wood affected by steel corrosion usually looks dark and soft. If the affected area is large relative to the area of the member, refer the condition to a competent engineer, as corrective measures will need to be taken.
- Check the cap where the beams bear directly on it and where the cap bears directly upon the piles. Note particularly any splitting or crushing in these areas. Distinguish between actual crushing, which involves some splintering, and compression resulting from moisture being squeezed out of the cells. Crushing reduces the strength of the member and affects its alignment. Compression only affects the alignment, but does not adversely affect strength.
- Observe caps that are under heavy loads for excessive deflections.
- Check for decayed or damaged timbers in the backwalls of end bents (abutments), especially where such conditions would allow earth to spill upon the caps or stringers.
- Check timber piles in salt water for damage by marine borers. Specifically, pay attention to checks in the wood, bolt holes, laps, or other connections.
- For dolphins, fenders, timber piles and other members, inspect the upper portions lying between the high water and mudline for marine insects and decay. Check the fender pieces exposed to collision forces for signs of wear.

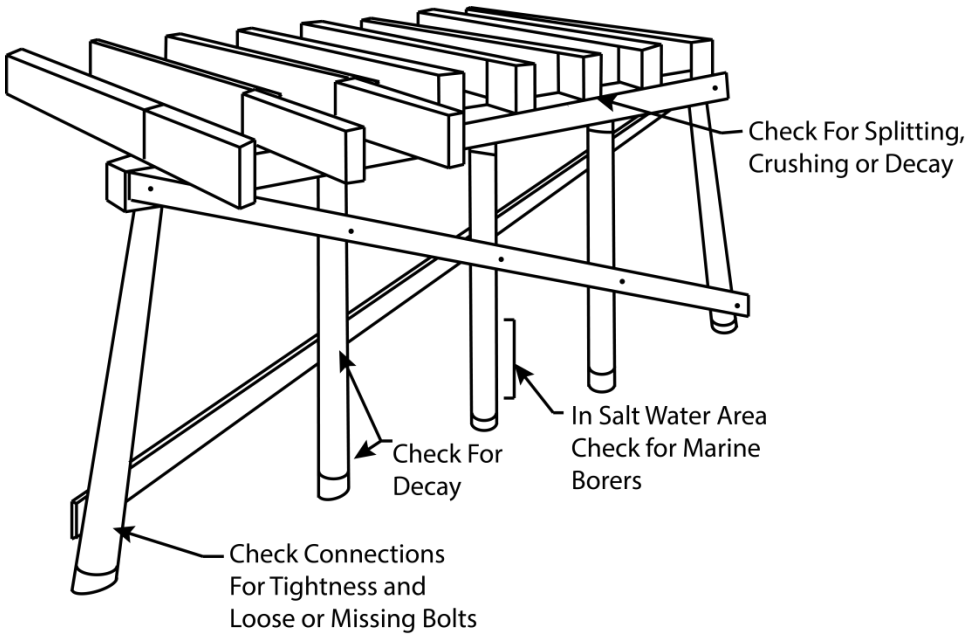


Figure 13.17 Common Timber Bent Inspection Items

It is essential that the area around timber pile bents and abutments is scalped and all weeds and brush removed from the vicinity because of the fire hazard. Where timber pilings are subject to frequent damage from ice, using discarded motor grader blades or other forms of armor should protect them.

Impact and overloading damage will be evident in the form of shattered or injured timbers, sagging or buckled members or timbers with large longitudinal cracks. The inspector should give the location and extent of damage and determine whether immediate remedial action is required.

The presence of water stains and mold/fungus is valuable information for identifying areas of high moisture content. It is also important to visually examine fastener connections as these joints may have undergone fatigue or because they are inherently areas of high moisture content. Piles and other parts of the bridge subject to moving water should be examined for mechanical damage due to water and debris or vessel impacts. The presence of moisture may lead to wood rot, which will degrade the strength of the wood. As moisture contents may vary seasonally, it is important that the appearance of water stains be used to identify potential sites of decay.

A chipping hammer, an ice pick, and an increment borer are the primary tools used for assessment of wood deterioration. The soundness of all timbers should be first checked by tapping with the hammer. Hammer sounding has been done for centuries and is still used today. The procedure is to hit a structural wood member with a hammer and determine from the quality of the sound if the component is sound. A dull or hollow sound indicates problems and the need for further investigation whereas a ringing, clear sound indicates the wood is good. The accuracy of “hammering” depends on judgment, experience and hearing. Decay has to be extensive and/or near the surface for this method to be effective. Wood that splinters is usually good. Wood that breaks abruptly is usually decayed. An ice pick can be used to probe for soft spots.

Coring and boring are the most widely used destructive methods for detecting internal decay. It is very important that coring and borings are done with sharp instruments. If extensive damage is suspected, an increment borer should be used to take a test boring and fully define the extent of deterioration. The increment borer is a specialized tool used to extract a section of wood tissue from a timber element with relatively minor damage to the element. The tool consists of a handle, an auger bit, and a small, half circular, metal tray (core extractor) that fits into the auger bit. The auger bit is usually manufactured from carbide steel. The tool breaks down and the auger bit and extractor fit into the handle. This makes it highly compact and easy to carry. Boring permits direct examination of a sample of the inside of a member. Since decay starts on the interior of a treated timber member, this test is needed to determine if a member should be removed as part of a repair. It is important to clean bore drills after use so that they do not transfer decay organisms. Borings should be taken very selectively to not further weaken the already damaged timber, and borings should not be made at all if no deterioration is evidenced. Once the boring has been made, save the boring sample and make sure that plugs treated with the same treatment as the timber are inserted into the hole made by the borer.

Moisture meters measure the moisture content of timber members. Moisture meters give qualitative readings above 30 percent. These hand-held meters provide a rapid means of sampling moisture content of wood. The measurements must be compared against a calibration curve to obtain an indirect measure of moisture content. Preservatives such as creosote and adhesives for glue lam products affect readings significantly. Two types of meters are available: conductance meters and dielectric meters.

The shigometer measures electrical resistance to detect decay in wood. The device is inserted into drilled holes at various depths to measure resistance. Wood should have a uniform resistance throughout its cross-section. This device requires frequent calibration. Since it also requires drilling holes, it may be simpler to merely take bore samples or to use a shell thickness rod. Shigometers allow the user to detect the initial breakdown of wood by fungi. The wood being inspected has to have moisture content above 30 percent for this device to work.

A pilodyn is used to measure the soundness of wood surfaces. The tool has a spring-loaded probe that measures the shock resistance of wood. The depth of penetration of the probe into the wood is an indication of its soundness.

Impact-echo detects low-density regions inside timber. Low-density regions are most likely decayed. The test is useful on Douglas fir and western red cedar but is not useful for southern pine.

Infrared thermography (IR) can be used, in the right circumstances to determine if wood has an excessive amount of moisture within it. The limitation to IR thermography is that the equipment is expensive, the operator must be well-trained and the conditions must be just right for the temperature difference to occur. Another limitation is that IR thermography provides only an inference about the moisture content of the wood, rather than a direct measure of either wood rot or residual strength.

In-place strength of timber members can be determined by ultrasonic testing. The load carrying capacity of a member is related to the wave velocity normal to the grain and its weight. This test should only be performed by experienced personnel.

Insect damage is evident by observing the damage to the timber. Insects that may cause damage include:

- Termites: All damage is inside the surfaces of the wood; hence, it is not visible. White mud, shelter tubes or runways extending up from the earth to the wood and on the side of masonry substructures are the only visible signs of infestation. If the timber members sag excessively, check for termite damage with an ice pick or an increment borer.
- Powder-post Beetles: The outer surface is poked with small holes. Often a powdery dust is dislodged from the holes. The inside may be completely excavated.
- Carpenter Ants: Accumulation of sawdust on the ground at the base of the timber is an indicator. The large, black ants may be seen in the vicinity of the infested wood.
- Marine Borers: Marine borers commonly enter the wood through bruises, breaks, or unplugged bolt holes. The area of infestation generally runs from the mudline to the water level at high tide.
- Mollusk Borers (Shipworms): The shipworm is one of the most serious enemies of marine timber installations. The most common species of shipworms is the teredo. This shipworm enters the timber in the early stage of life and remains there for the rest of its life. Teredos reach a length of 15 inches and a diameter of 3/8 inch, although some species of shipworm grow to a length of 6 feet. The teredo maintains a small opening in the surface of the wood to obtain nourishment from the seawater.
- Crustacean Borer: The most commonly encountered crustacean borer is the limnoria or wood louse. It bores into the surface of the wood to a shallow depth. Wave action or floating debris breaks down the thin shell of timber outside the borers' burrows, causing the limnoria to burrow deeper. The continuous burrowing results in a progressive deterioration of the timber pile cross line. In such cases, there are often no outside evidences of borer attack.

The most common treatment for dry land insects is to pressure treat the wood with a proper material, and/or replace damaged member with new pressure treated member, and keep careful watch for reinfestation.

The best method of control of marine borers is prevention of infestation: 1) prevent bruises by installing buttresses to protect the structural members from damage by wave action, current flow, or floating debris, and 2) treat damaged wood by plugging or coating all holes, bruises, and freshly sawed ends with heavy creosote.

Fungi usually require some moisture to exist. As a rule, fungus decay can be avoided only by excellent preservative treatment. Fungus decay is classified as follows:

- Mild fungus decay appears as a stain or discoloration. It is hard to detect and even harder to distinguish between decay fungi and staining fungi.
- Advanced. Wood darkens further and shows signs of definite disintegration, with the surface becoming soft and spongy, stringy, or crumbly, depending upon the type of decay or fungus. Fruiting bodies of fungi, similar to those seen on old stumps, may develop. The inspector should note the location, depth of penetration, and size of the areas of decay. Where decay occurs at a joint or splice, the effect on the strength of the connection should be indicated. A knife, icepick, or an increment borer can be used to test for decayed wood.

13.3.1 Cleaning

Cleaning dirt and debris from the deck surface, drains, and other horizontal components also reduces moisture and improves air circulation. Accumulated debris should be removed from any timber surface. Debris holds moisture that will penetrate into the timber member. The bark of native logs should be removed if it is not removed during construction. This prevents moisture from being trapped between the wood and bark.

Pre-treated timber will deteriorate after about 5 years if it is not maintained. Pressure washing and sealing exposed timber elements should be done on a routine basis. The maintenance interval should be selected in accordance with the manufacturer's instructions.

13.3.2 Moisture Protection

Moisture content of wood is one of the most significant factors regarding wood decay because a considerable amount of water is required for fungus growth and in many instances, the moisture content of wood in service is subject to control. Moisture control was the only method used for protecting many covered bridges constructed of untreated timber, some of which have provided service lives of 100 years or more. Moisture control is the simplest, most economical method of reducing or preventing decay in timber bridges. When exposure to wetting is reduced, members can dry to moisture contents below that which will support most fungal and insect growth (approximately 25 percent).

Not only does the amount of water in wood directly control the possibility of fungus infection and growth, but it also is significant to the decay process in less direct ways. Prolonged or repeated wettings contribute to leaching and a consequent loss of natural decay resistance. Further, during the seasoning of large timbers, the loss of water is accompanied by shrinkage that normally results in the development of seasoning checks. Such checks may expose untreated parts of preservative-treated timbers and may also form water-trapping pockets that can become infection sites for decay fungi.

Decay is very likely to occur at connections, splices, support points, around bolt holes, and at the ends of field cut members (such as beams, pile tops, or chords). This can be attributed to the tendency of such areas to collect and retain moisture. Unless these surfaces are protected, decay is very likely. Although modem timber bridges are protected with preservatives, decay can still occur in areas where the preservative layer is shallow or broken. Many times pressure treated lumber may be field cut exposing the interior of the members to a decay entry point. This damage is the major cause of deterioration in timber bridges.

Moisture control involves a common sense approach of identifying areas with visibly high moisture contents, locating the source of water, and taking corrective action to eliminate the source. For example, drainage patterns on approach roadways can be rerouted to channel water away from the bridge rather than onto the bridge. New cuts and bolt holes in treated timber should be thoroughly coated with preservative materials to prevent moisture from entering the wood and causing decay.

Whenever repair or rehabilitation of a timber structure is done, all effort should be made to include field-treatment of holes and cuts made as part of the work performed.

Some attempts at moisture control such as the use of roll roofing material as stringer covers under decking or as piling top covers under caps or sheet metal as water-diverting cover or caps

may contribute to increased wetting. As roofing material ages, it becomes permeable to water and the material will inhibit the drying.

If a drift pin is used to install metal sheeting over the pile, a slight depression in sheet metal is formed and a less than watertight seal is created. This results in an avenue for water to infiltrate the timber. Nails through the metal into the pile top function similarly. As a result, banding is preferred.

13.3.3 Preservation Treatments

Most untreated wood will decompose when the four conditions required for decay and insect attack occur:

- High moisture
- Favorable temperature
- Oxygen
- Food source (wood fiber)

If any one of these conditions is removed, infestation and decomposition cannot occur. Eliminating wood fiber as a food source by using pressure-treated wood products is an easy and practical solution. Research has shown that wood can be expected to last for many decades when properly treated and installed for its intended use. The map in Figure 13.18 indicates, by region, the degree of potential wood deterioration. Because deterioration zones range from moderate to severe across most of the country, it is important to understand the types of wood treatments available for use in timber structures.

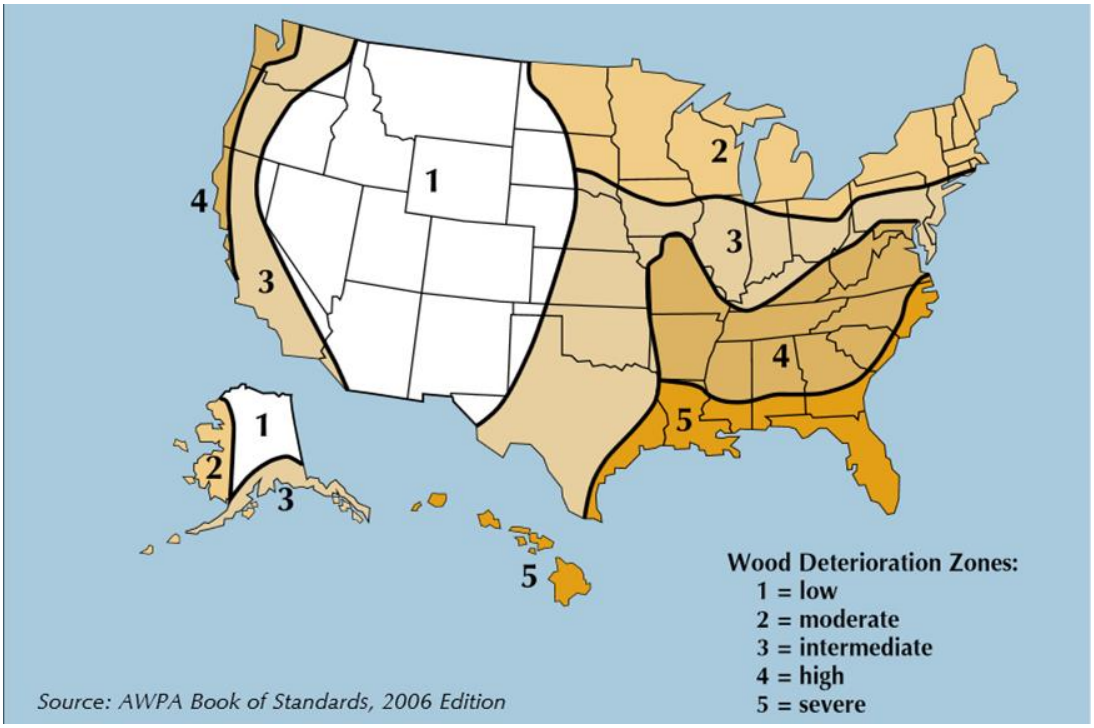


Figure 13.18 Potential for Wood Deterioration Throughout the United States

Types of wood preservation to be discussed in this section include:

- Surface treatments (bridge element replacement)

- Pressure treatments (at the plant)
- In-Place treatments (in the field, includes surface treatments, internal treatments, and wrapping)

Timber bridges use pressure treated wood since it provides the best long term protection. The additional life obtained by pressure treatment over that of untreated wood will be affected greatly by the level of penetration and retention of preservative in the wood and conditions of service. For most pressure treated wood, whether used above or in ground contact applications, data indicates that when properly installed and maintained, the expected service life could be in excess of 30 years for most applications.

When the pressure treated wood is brought out to the bridge site, it may have to be cut to size and drilled through during installation. Figure 13.19 shows an existing timber deck being replaced. Cutting and drilling through the pressure treated wood exposes internal areas of the wood where the treatment may not have penetrated. For this reason, surface treatments are applied in the field.

Figure 13.19 New Bridge Replacement Elements Need Surface Treatment

The effectiveness of surface treatments depends on the thoroughness of application, wood species, size, and moisture content at the time of treatment. Wet wood absorbs less preservative than does dry wood. This factor is significant in timber bridges because many areas requiring treatment are subject to wetting. Tests indicate that improved treatment of wet wood was obtained by using preservatives at double the normal 3- to 5-percent concentration. Although field tests show that surface treatments in above ground locations can prevent decay infections for up to 20 years or more, it is recommended that treatments used for bridge applications be systematically reapplied at intervals of 3 to 5 years to ensure adequate protection from decay.

The American Wood Protection Association (AWPA) is the principal standards-writing body for wood preservation in the United States. The AWPA Book of Standards establishes what preservatives and chemical formulations are appropriate for common applications; sets treating procedures; establishes wood species requirements and testing procedures; and provides guidance on quality control and inspection.

13.3.3.1 Surface Treatment (Bridge Element Replacement)

If a timber bridge element needs to be replaced, its removal can potentially expose untreated surfaces of existing timber elements. Surface treatments are not generally as satisfactory as those of pressure treatment in achieving optimum protection and extended service life. The processes do serve a useful purpose when more thorough treatments are impractical or exposure conditions are such that little preservative protection is required. Non-pressure methods, in general, consist of (a) surface application of preservatives by brushing or brief dipping,(b) soaking in preservative oils or steeping in solutions of waterborne preservatives, (c) diffusion processes with water-borne preservatives, (d) vacuum treatment, and (e) a variety of miscellaneous processes. Surface treatments are applied to these existing bridge members to protect newly exposed, untreated wood from decay or to supplement the initial treatment some years after installation. They can be applied in liquid, power, or solid form. This type of treatment is most effective when applied before decay begins and is commonly used for treating checks, splits, delaminations, mechanical damage, or areas that were field-fabricated during construction. The ease of application and effectiveness of surface treatments as barrier make them useful in preventive maintenance. However, the shallow penetration limits their effectiveness.

During timber element replacement anchor bolts, drift pins, and lag bolts create holes where decay and deterioration often begins. These holes should be protected from moisture penetration by swabbing with hot asphalt or treating with a preservative using a pressure bolt hole technique. This technique uses a preapproved device called a bolt treater to apply a pressure to the circumference of the bolt hole while applying the preservative. Unplugged holes such as those left by test borings, nails, bolts, and the like, also permit entrance of pests. Their location and extent should be noted and recommendations should be made for their repair.

The natural opening in the grain of timber ends allows easier water penetration. If the end is cut, it should be painted with a preservative or swabbed with hot asphalt. The ends of the exposed members should be capped with a thin sheet of aluminum, tin, or similar material.

Figure 13.20 shows a timber bridge deck being replaced. Note the drills and saws that cut through the pressure treated lumber.

Figure 13.20 Timber Bridge Deck Being Replaced

13.3.3.2 Pressure Treatment (at the Plant)

In commercial practice, wood is most often treated by immersing it in a preservative in a high pressure apparatus and applying pressure to drive the preservative into the wood. The chemical preservatives react with the wood fiber to form a treated wood product resistant to attack by insects, decay, fungus and marine borers. Reaction of the chemical preservative within the wood's fiber begins during the treating cycle. The time needed to maximize fixation or stabilization of the preservative in the wood fibers can range from several hours to several days, depending on ambient temperatures and humidity that vary greatly with locale and seasonal conditions and/or various post treatment processes that might be employed by the treating facility

Treated wood products in compliance with AWPA standards are labeled with a plastic tag, called a quality mark, usually stapled on the end to the timber. The mark indicates a third party inspection has been completed, adhering to the AWPA standards of quality. Information on the quality mark includes the AWPA recommendations on the use, exposure categories for the timber, and identifies the preservative used and the moisture content after treatment. This is useful information for preservation and maintenance actions involving pressure treated wood. An example quality mark and definitions are shown in Figure 13.21 below.

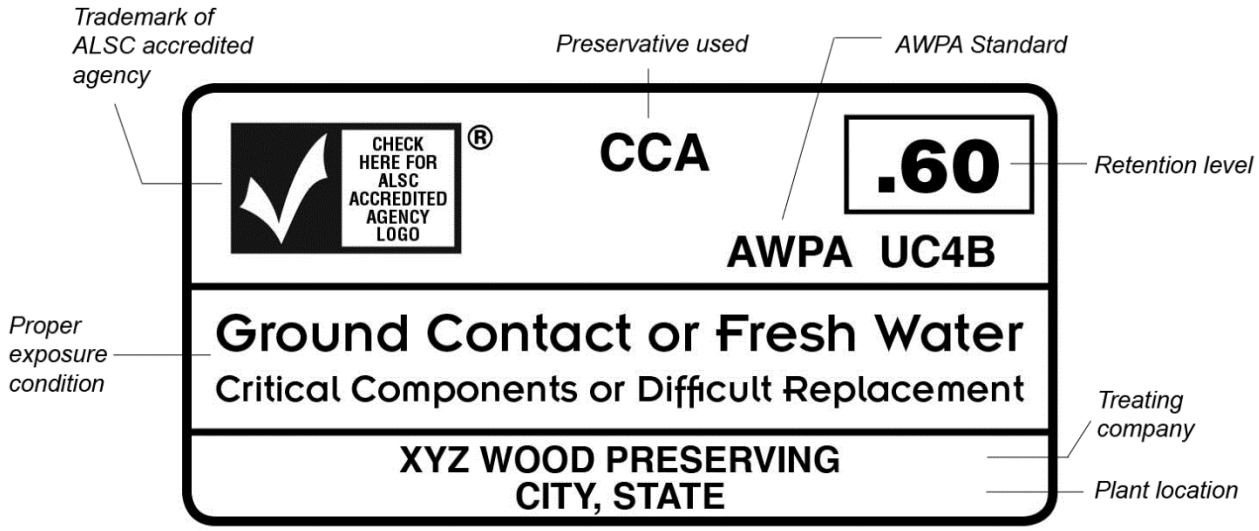


Figure 13.21 Interpreting a Quality Mark for Timber

There are three broad classes of preservatives used for the pressure treatment of wood products:

1. Waterborne preservatives, where water is the carrier for the preservative chemicals, serve a wide variety of uses. These include residential, commercial, marine, agricultural, recreational and industrial applications.
2. Oil-type preservatives are used primarily for industrial applications including utility poles, piling, posts, glulam beams and timbers.
3. Creosote preservatives, including creosote/coal-tar mixtures, protect railroad ties, marine pilings and utility poles.

For decades, treatments for industrial products were dominated by Chromated Copper Arsenate (CCA), creosote and pentachlorophenol. Although these traditional preservatives continue to be widely used, newer alternative treatments are becoming increasingly available.

Waterborne preservatives are commonly specified for most residential, commercial and marine building applications, including timber bridges. Waterborne treatments are clean in appearance, odorless and paintable.
They are also EPA-registered for both interior and exterior use without a sealer.

Incising can be done for wood species that do not allow preservation treatments to consistently and uniformly penetrate into the fibers. Incising is a mechanical process wherein numerous longitudinal incisions are made with chisel-type or knife-type teeth into the wood surfaces, parallel to the grain direction. Incising increases preservative retention and penetration during the treating process by increasing the amount of exposed, easily penetrated end-grain and by increasing the side-grain surface area.

While preservative treatment increases durability, the incising process is known to reduce certain strength values of structural lumber caused by the crushing of wood fibers. The type and amount of strength reduction is dependent on the depth of incision. Research results found the depth of incision to be the critical factor on determining the reduction of mechanical properties.

Treated wood must meet minimum preservative penetration and retention requirements for use in a particular service condition. Penetration refers to the depth a preservative must permeate into the wood fiber. The amount of preservative that remains in the wood after the pressure-treating process is complete is called retention. Retention levels are expressed in pounds of preservative per cubic foot (pcf) of wood fiber and the higher the retention, the harsher the service condition the wood may be exposed to.

Creosote pressure treatment is the most effective method of protection for bridge timbers. But its use is also highly restricted by environmental laws. Pentachlorophenol (penta-oil treatment) is a heavy oil solvent applied using pressure methods. Inorganic salt solutions are applied using pressure and provide less water repelling than other treatments. The salt solution can corrode any hardware used to construct the bridge. For more information on hardware, see Section 13.3.3.3 Connections in Preservative Treated Lumber.

13.3.3.3 Connections in Preservative Treated Timber

Fasteners and connectors for preservative-treated wood need to be hot-dipped galvanized in accordance with ASTM A-153, silicon bronze, copper or 304 or 316 stainless steel. Stainless steel fasteners should be used below grade and are recommended for use with treated wood in other corrosive exposures such as in or near salt water.

Hot-dipped galvanized or stainless steel fasteners and connectors are recommended for use when lumber is treated with a copper-based preservative. Copper-based formulations include CCA, ACZA, ACQ, or CA-C and may be used in interior or exterior applications. Hot-dipped galvanized fasteners and connectors are generally acceptable for above-grade applications. Fasteners and connectors used together must be of the same metallic composition to avoid galvanic corrosion (e.g., use hot-dipped nails with hot-dipped joist hangers). Type 304 or 316 stainless steel is recommended for maximum corrosion resistance in more severe exterior

applications including, salt-water exposure and below-grade applications. Stainless steel is also a recommended option for use with CCA, ACZA, ACQ, or CA-C treated wood at retention levels greater than required for ground contact.

Standard carbon-steel or aluminum products should never be used in direct contact when lumber is treated with a copper-based preservative. Electroplated galvanized metal products generally have a thinner layer of protection compared to hot-dipped galvanized and are typically not accepted by the building codes for use in exterior applications. When aluminum or electroplated products such as flashing or termite shields are used, spacers or other physical barriers are necessary to prevent direct contact from copper-based treated wood. These barriers should provide complete separation and remain intact for the intended service life of the metal.

13.3.3.4 In-Place Preservative Treatment (Surface Treatments and Internal Treatments)

In-place treating can provide a safe, effective, and economical method for extending the service life of timber bridges. Most of the techniques and treatments were developed for use on railroads or utility poles, for which they have been used effectively for many years. A large number of timber bridges have been treated in-place, extending service life by as much as 20 years or more.

Surface Treatments

To further protect in-service wood timbers, sealants can be applied to prevent infection of exposed wood. The use of bituminous or asphaltic mastics as sealants or bedding compounds can be effective sealants. Copper naphthenate-based solutions may be used for field treatment of materials originally treated with pentachlorophenol, creosote or waterborne preservatives as specified in AWPA Standard M4. The preservative concentration should contain no less than 2 percent copper metal. Newly exposed surfaces resulting from field fabrication and/or handling abuse should be field treated by brushing, dipping or soaking. The application should be done in a manner that the preservative does not drip or spill into the surrounding soil.

Internal Treatments

Internal treatments usually involve either water-diffusible or fumigants. The most popular and recommended water-diffusible treatment are boron based and are discussed in detail in Section 11.3.3.1 of this manual. Fumigants are discussed below but are being used less for timber bridge preservation.

If decay is already occurring, fumigants are used. Fumigants are preservative chemicals available in liquid or solid form that can be used to control internal decay from fungi and insects. Fumigants are placed in prebored holes to arrest internal decay. Over a period of time, the fumigants volatilize into toxic gases that move through the wood, eliminating decay fungi and insects. Fumigants can diffuse in the direction of the wood grain for 8 feet or more from the point of application in vertical members, such as poles. In horizontal members, the distance of movement is approximately 2 to 4 feet from the point of application.

The three chemicals most commonly used as liquid fumigants are Vapam (33-percent sodium N-methyldithiocarbamate), Vorlex (20-percent methylisothiocyanate, 80-percent chlorinated

C3 hydrocarbons), and chloropicrin (trichloronitromethane). Solid fumigants are available in capsules of methylisothiocyanate (MIT), which is the active ingredient of Vapam and Vorlex.

Solid fumigants provide increased safety, reduce the risk of environmental contamination, and permit fumigant use in previously restricted applications. Borate has become the treatment of choice for internal solids, see Section 11.3.3.1 for more information.

To be most effective, fumigants must be applied to sound wood. When applied in very porous wood or close to surfaces, some of the fumigant is lost by diffusion to the atmosphere. Before applying fumigants, the condition of the member should be carefully assessed to identify the optimal boring pattern that avoids fasteners, seasoning checks, badly decayed entire structure, it is advisable to contract the project to specialists in the field.

Protection of Piles by Wrapping

Additional protection should be applied to timber bridge piles at the groundline since this is where the deterioration will occur first. Installation of a PVC barrier is a typical protection measure.

A suggested procedure for pile protection follows and is illustrated in Figure 13.22:

Suggested Procedure

Pile Protection

1. Piles should be protected 1 foot below the mudline and 1 foot above high tide.
2. A flexible PVC barrier can be installed when a pile loses approximately 10 to 15 percent of its cross- sectional area.
3. Sheath the pile with a 30-mil PVC sheet. A half-round wood pole piece is attached to the vertical edge of the PVC sheet to help in the wrapping process. (Note: A pile with creosote bleeding from its surface must first be wrapped with a sheet of polyethylene film prior to installing the PVC wrap to prevent a reaction between the PVC and the preservative.)
4. Staple lengths of polyethylene foam, ½ by 3 inches, about 1 inch from the upper and lower horizontal edges of the sheet.
5. Fit the pole pieces together with one inserted into a pocket attached to the bottom of the other pole.
6. Roll the excess material onto the combined pole pieces and tighten around the pile with a special wrench.
7. Secure the wrap and poles with aluminum alloy rails.
8. Nail rigid plastic bands at the top and bottom directly over where the polyethylene foam is located under the wrap.
9. Install additional bands on equal distance centers between the top and bottom bands.

For timber elements other than piles, wrapping with PVC sheeting is an effective preservation action. Examples include braces or other timber elements with slight damage or to prevent

damage from occurring. Remove the bolt which secures the brace to the pile. Wrap the freed end with 20-mil flexible PVC sheeting. Drive the bolt through the wrapping and existing hole. Rebolt the brace to the pile. Wrap the remainder of the brace in the sheeting.

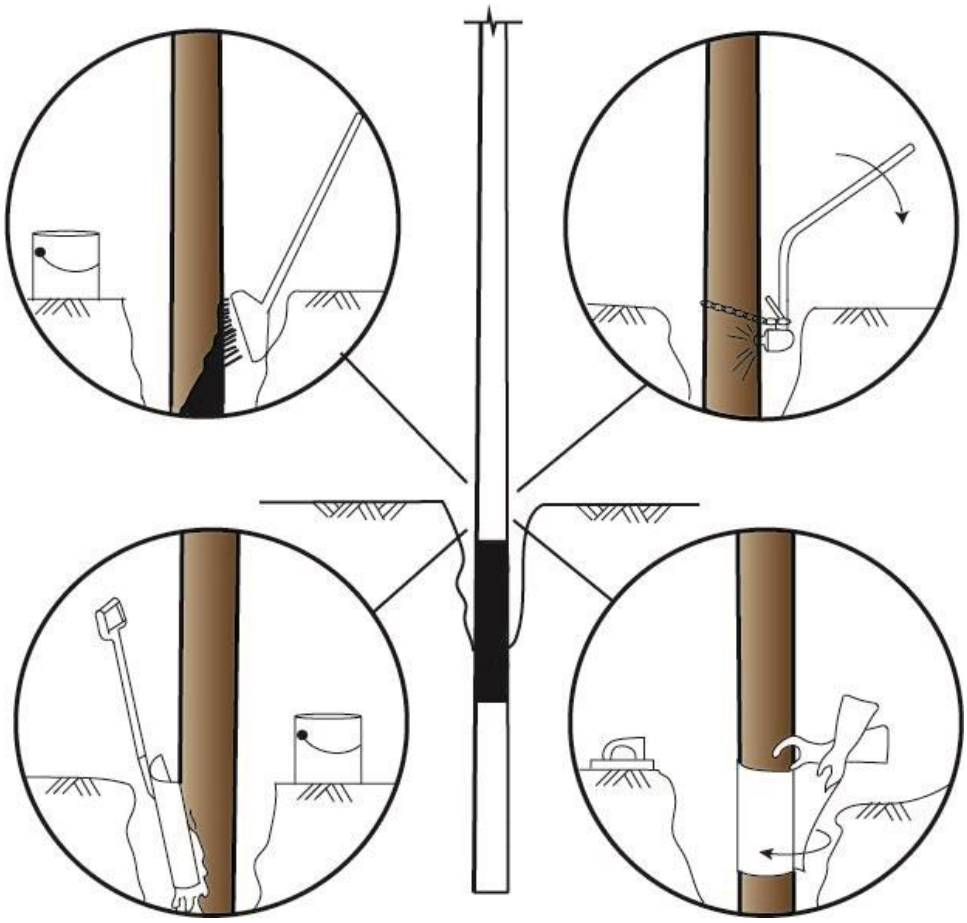


Figure 13.22 Methods of Applying Groundline Treatment to Bridge Pile Bents

Protection of Piles Tops

As bridge pile bents are initially constructed or replaced they need to be cut to the correct elevation for the cap. When this happens the top of the freshly cut pile is susceptible to deterioration if not treated.

Two options exist for treatment of pile ends: (1) Drill 3/4-inch holes evenly spaced into the pile top 1½ inches deep, fill holes with a preservative, and cap with aluminum sheeting. (2) Clamp an iron ring around the top of the pile, pour a preservative into the ring, and allow the pile to absorb the preservative, remove the ring, and cap the pile. Rounding the top of the pile helps to shed water off the horizontal surface.

Preservative is injected into the pole at the groundline with a special tool or applied on the pole surface as a paste or bandage.

13.3.3.5 Handling and Disposal of Treated Wood

When applying use of pressure-treated wood, do not use where the wood will be in frequent or prolonged contact with bare skin or under circumstances where preservative may become a component of food for either humans or animals. Examples include sites using mulch from recycled treated wood, cutting boards, counter tops, animal bedding and structures for storing human or animal food. Treated wood should not be used where it may come into contact or indirect contact with public drinking water except for uses such as docks and bridges. Wood treated with Pentachlorophenol or Creosote should not be used where it may come into contact with drinking water for domestic animals or livestock. Waterborne preservatives are approved for this use.

Recommendation

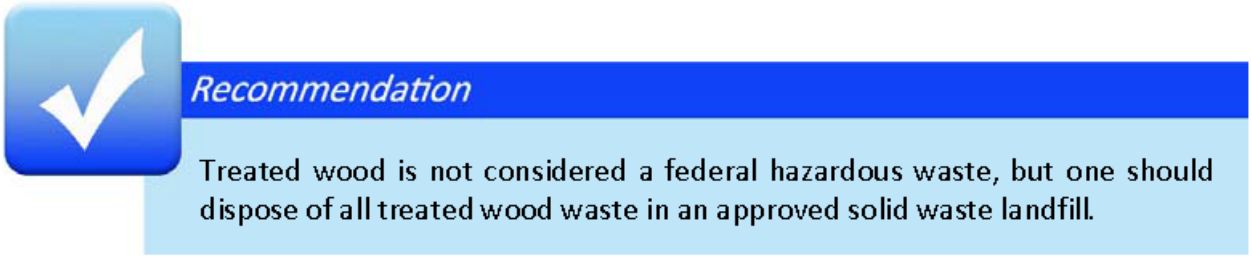
When handling pressure treated wood, take proper safety precautions.

When handling treated wood, wear protective clothing such as long-sleeved shirts and long pants and use gloves. After working with the treated material, wash any exposed area before eating, drinking, going to the toilet, or using tobacco products. When sawing and machining the treated material, wear goggles to protect eyes from flying particles. Wear a dust mask and, if possible, work outdoors to avoid inhalation of sawdust.

Under the Federal Insecticide, Fungicide and Rodenticide Act (FIFRA), wood preservative systems containing chemical pesticides are fully regulated by the U.S. Environmental Protection Agency (EPA) and are required to undergo a rigorous registration and re-registration process. The EPA considers wood preservative systems as antimicrobial pesticides and requires that the pesticides be supported with thorough scientific review and analysis, as well as show they can be used in wood preservative products without causing undue adverse effect to human health or the environment. Further, under federal law, preserved wood products themselves are not considered to be a pesticide and therefore are not regulated by FIFRA. Further, under the Resource Conservation and Recovery Act (RCRA), which regulates waste disposal, treated wood is not listed as a federal hazardous waste. Treated wood is also exempt from state hazardous waste regulations. Being a non-hazardous waste allows material deemed to be waste treated wood to be disposed in an approved state solid waste landfill or properly permitted combustion facility. Treated wood would only be regulated as a “hazardous waste” if (1) it was to become listed in the RCRA regulations, or (2) the material exhibited a hazardous waste characteristic. Studies have determined that treated wood does not exhibit a federal hazardous waste characteristic.

If material removed from service is not designated as waste treated wood, the preferred option for handling is to reuse the material in a manner consistent with the use of similar treated wood products. Material originally used for structural applications can often either be used for structural purposes if found to still be sound or for non-structural purposes such as landscaping timbers or parking bumpers, or posts. Treated wood should NEVER be burned in open fires of any kind, stoves, fireplaces, or residential boilers.

Treated wood can be used in an industrial biofuel burner, if it is a permitted facility meeting state and federal air quality standards. Treated wood is also considered biomass in some states and may be combusted for generation of energy and/or be eligible for renewable energy credits. For example, creosote treated timber products have a long history of being used as a renewable resource for the generation of energy through combustion. As a last resort, all treated wood waste, whether it is generated from commercial, industrial, or residential uses, can be disposed of as non-hazardous material in an approved solid waste landfill, provided it is disposed in conformance with federal, state, and local regulations.


Recommendation
Treated wood is not considered a federal hazardous waste, but one should dispose of all treated wood waste in an approved solid waste landfill.

During construction or installation of treated wood products, it is important to collect all scraps, sawdust, and curing debris and remove for appropriate disposal, especially when the area is readily accessible to the public or near and over bodies of water.

13.4 Repair and Rehabilitation of Concrete Substructures

Problems often found in reinforced concrete substructures include the deterioration of concrete and the corrosion of reinforcing bars. The problems that deteriorate other reinforced concrete elements are generally the same problems that create problems in the substructure. Repairs to the substructures generally can be performed using the same basic materials and techniques as used for all reinforced concrete. The repairs however, may involve supports and shoring of the superstructure and deck and unlike for deck repairs are more often done on vertical surfaces. Specific repairs involving piles and pile bents will be presented later in this chapter. As all the loading is carried by the substructure, it is critical that the substructure is maintained in a state of good repair.

Substructures deterioration initiates similar to any other reinforced concrete bridge elements. It can begin with the failure of other elements, such as moisture and contaminants falling through leaking deck joints, environmental or climatic conditions, such as substructures in a marine environment or salt spray onto piers near the shoulders of underpasses, restricted movement, as when frozen bearings create thermal forces and pressure is created on substructure elements, or impact damage. Lateral force such as large debris striking a bridge during periods of high water or an over-height vehicle hitting a beam can also create large forces on the anchor bolts which in turn are transmitted to the substructure cap, which can cause damage to the bridge seats or cracks in other parts of the substructure such as the columns. These problems can be traced back to inadequate design, improper placement of the assemblies, movement of the superstructure, or corrosion related friction between the sliding surfaces.

During the preliminary planning stage, necessary substructure repair procedures should be determined. One of the first questions to ask is whether the repair intends to only protect the element from additional damage or is it intended to improve or restore its load carrying

capacity? These procedures should then be scheduled in a logical order, and they may include the following:

- Identify damage by sounding and marking the unsound concrete
- Make provisions to correct the cause of damage
- Determine whether the repairs are structural or non-structural

Structural repair of substructure elements may require the loads to be temporarily supported by shoring while the repair is performed and the repair concrete reaches full strength. If the deterioration is caused by loads, or in case of extensive concrete deterioration, the superstructure may need to be lifted (to take the load off the substructure) prior to repair. The repair may also require raising the superstructure in order to provide workspace. Shoring and temporary supports are discussed at the end of this chapter.

When to Call the Engineer

Call the Engineer if temporary support of loads or raising of the superstructure is required to perform the repair.

13.4.1 Surface Repairs

Surface repairs are generally non-structural repairs. In most cases, rebar corrosion has caused the facing concrete to deteriorate and spall. The concrete behind the rebar remains sound and capable of carrying the loads. Surface repairs can be done without shoring and temporary supports. The work can be done while the bridge remains in service, though traffic may have to be moved away from the immediate area of the repair.

The column repair shown in Figure 13.23 was performed under loading as the repairs were only to the superficial concrete covering the rebars.

Figure 13.23 Non-Structural Repair of a Column

Substructure concrete will deteriorate from the effects of water, deicing chemicals, freeze cracking, settlement cracking, structural cracking, or impact by debris. The deterioration leads to spalling and results in edges or portions of the cover concrete breaking off. This condition requires that repairs be made to prevent continued deterioration, specifically additional spalling as moisture continues to reach the rebar causing additional corrosion. Surface repair procedures are often used to face old rubble masonry or concrete made from large stone. Superficial substructure damage can be repaired without affecting traffic. Concrete is removed, a protective coating can be applied to the rebar, and the repair material can be placed by conventional form and pour methods or shotcrete.

Deterioration at the waterline is particular to abutments or piers in streams. It forms a depression or cavity in the concrete extending some distance above and below the average waterline of the stream. Deterioration at the waterline usually occurs on the upstream face or along the sides of the pier. Repairs at the waterline are very similar to the surface deterioration problem except it is necessary to control the stream flow so that the work can be kept dry.

The following procedures can be used to repair deterioration of this type:

Suggested Procedure

Non-Structural Repair for Concrete Substructure Components

1. Dewater the abutment or pier.
2. Chip away all loose concrete in poor condition.
3. Chip or roughen the surface to provide a better bond between old and new concrete.
4. Clean the reinforcing bars of scale and loose rust using abrasive blast cleaning.
5. Clean the surface in all areas where new concrete will be placed.
6. Construct a form of adequate strength.
7. Apply a bonding agent to the surface (if owner allows)* and fill the form with concrete.

* Not all agencies authorize the use of bonding agents.

Effective bonding of the new concrete to the old is usually accomplished with a bonding material and is particularly important when deep deterioration requires a large volume of concrete to be replaced. Installing rebar into the existing concrete to act as anchorage for the repair material ensures the repair material will not fall out if the repair becomes de-bonded. A grout of neat cement base can be used as an effective bonding agent. Grout can also be used when the form for the concrete is so inaccessible that an epoxy material cannot be applied effectively. The exposed area can be liberally coated with grout just prior to pouring the concrete.

Before and after photos of surface repair of a retaining wall are shown in Figure 13.24 and Figure 13.25.

Figure 13.24 Preparation for Repair of Retaining Wall

Figure 13.25 Retaining Wall After Repairs

13.4.2 Structural Repairs

For more extensive repairs aimed at the rehabilitation of the substructure concrete elements, involving structural repairs, common equipment and materials utilized are listed below.

The necessary equipment:

- Cribbing, jacks, shoring equipment
- Air drill
- Tie screw or equivalent bolts
- Wood spacers, walers, etc.
- Reinforcing rods or steel
- Hand tools
- Concrete removal equipment
- Anchor bolts and anchors
- Plywood sheet forming
- Cement concrete

The following steps in the rehabilitation procedure are normally required:

Suggested Procedure

Concrete Structural Repair

1. Establish traffic control, if necessary.
2. Install erosion control material if original material missing or disturbed.
3. Install shoring and jack the superstructure to remove the loading on the element (always check with an engineer for shoring and jacking procedures).
4. Remove deteriorated concrete by chipping and blast cleaning.
5. Drill and set the tie screws and studs to support the formwork.
6. Set reinforcing steel and forms.
7. Apply epoxy-bonding agent (if owner allows)* to the concrete surface just before placing the concrete.
8. Place the cement concrete, cure, and remove the forms.
9. Place superstructure back on newly repaired element and remove shoring.
10. Remove erosion control material and clean up the site.

* Not all agencies authorize the use of bonding agents.

An example drawing for more extensive repairs aimed at the rehabilitation of the substructure concrete elements is shown in Figure 13.26.

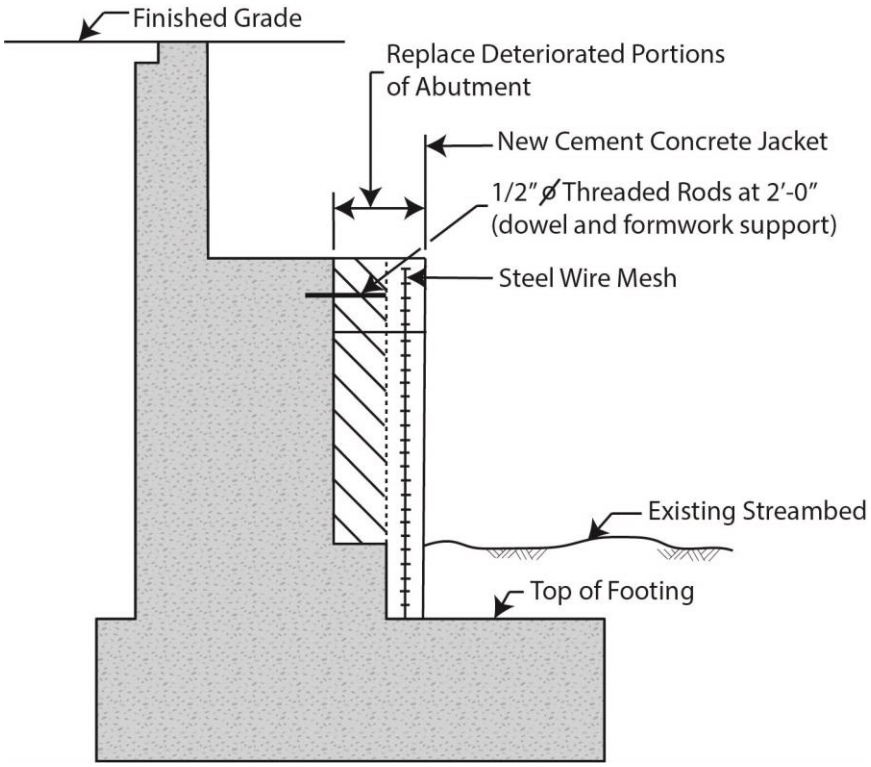

Finished Grade
Replace Deteriorated Portions of Abutment
New Cement Concrete Jacket
1/2" ø Threaded Rods at 2'-0" (dowel and formwork support)
Steel Wire Mesh
Existing Streambed
Top of Footing

Figure 13.26 Repair of Abutment Face

Portions of an otherwise sound concrete may be broken off by frost heave, ice that forms in voids created by fill settlement adjacent to the wall, ice in the cracks, voids in the concrete, or

insufficient air entrainment voids. Deterioration may occur due to deicing, salt-rich snow and ice plowed onto the substructure and retained by moisture-holding debris. Bad aggregates sometimes cause concrete failure; see Chapter 5 for further information on aggregate issues such as Alkali Silica Reactivity (ASR). The loss of portions of a substructure element can result in erosion of the fill and further damage to the wing wall and the approach.

The cause of the failure should be determined so that it may be corrected if possible and to ensure that any defects or deteriorated areas present can be removed in order to ensure an effective repair. The forming should be preplanned and the materials cut to size in advance if feasible. Any excavation required to gain sufficient working access and to facilitate the removal of defective concrete could be accomplished in advance of the repair.

Wingwalls

The wingwall repair consists of recasting the broken or deteriorated section as follows:

Suggested Procedure

Wingwall Repair

1. Excavate as required to set the dowels for formwork support.
2. Remove all fractured or deteriorated concrete to sound concrete by chipping, and blast clean to remove material left after chipping.
3. Drill and set dowels. Dowels, #4 bars, are placed a minimum of 9 inches into sound concrete and set with non-shrink grout, 18 inches on center.
4. Set the dowels for formwork support and install the forms.
5. Just prior to placing the concrete, apply an epoxy-bonding agent (if owner allows)* to all existing concrete that is to come into contact with new concrete.
6. Cure concrete for a minimum of 7 days before backfilling with granular material, or until concrete has developed sufficient strength to resist the imposed lateral pressures.

* Not all agencies authorize the use of bonding agents.

An example of a wingwall repair procedure is presented in Figure 13.27.

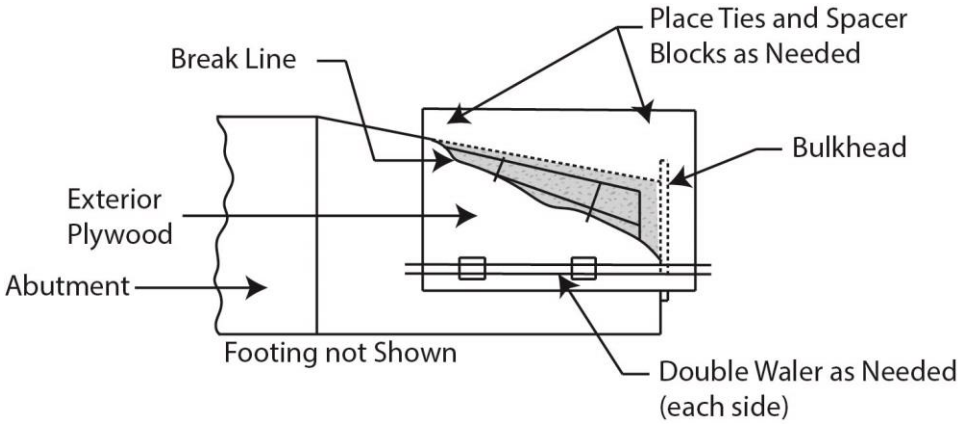


Figure 13.27 Reconstruction of Wingwall

Abutment Backwalls

Abutment backwalls can be damaged from several factors including shoving of the approach slab, wheel impact of vehicles from the joint, and salt contaminated run off from the joint. Damaged abutment backwalls may be partially or totally replaced.

Suggested Procedure

Abutment Backwall Replacement

1. If a steel armored joint is used, temporarily tack weld the abutment side of the steel armored joint to the deck side of the joint assembly. Certified personnel should perform welding.
2. Cut and excavate the approach slab or pavement to allow access to the backwall.
3. Remove deteriorated concrete from the backwall and clean concrete and reinforcing bars using abrasive blast cleaning.
4. Place replacement bars by lapping them with the existing bars or by drilling and grouting.
5. Place forms for concrete. Just prior to placing the concrete, apply an epoxy-bonding agent (if owner allows)* to all existing concrete that is to come into contact with new concrete.
6. Place and cure concrete. Insure concrete is placed beneath existing joints.
7. Remove forms and temporary tack
8. on joints.
9. Backfill and compact subgrade under the approach slab or pavement.
10. Patch the approach slab or pavement.

* Not all agencies authorize the use of bonding agents.

Pier Caps

Pier caps are exposed to water and chlorides from leaking joints. Debris retains the moisture and chlorides and if not removed, subjects the cap to continuous exposure to these contaminates. Corrosion of the main longitudinal steel may cause the concrete cover to spall

exposing the reinforcement to further corrosion. A horizontal crack along the face of the pier cap, 3 to 4 inches from the top, normally indicates that the top mat of rebars has expanded because of corrosion and has forced up (delaminated) the concrete. It is not uncommon to see a similar crack along the bottom of the cap.

A pier cap functions similar to a continuous beam, and repair considerations are similar. However, the method of repair and costs may be quite different depending on whether the repair can be performed in the dry or not. It may be possible to repair in the dry by scheduling for a certain time of year. Cofferdams may be possible depending on the streambed material, water depth, water velocity, and environmental requirements.

When to Call the Engineer

Call the Engineer if pier cap has vertical cracks at midspan between columns, emanating from bottom of cap, or shear cracks in cantilever areas of hammer head pier caps.

Figure 13.28 Pier Cap and Column in Need of Structural Rehabilitation

Spread Footings

Deterioration of concrete in spread footings can be caused by any of a number of corrosive chemicals which are often found in soils or groundwater. Geotechnical investigations should include evaluations of the presence of these types of chemicals. If they are found to be present, appropriate protective measures should be taken. A wide range of options exists including the use of special materials (or additives to standard materials), protective surface treatments and more frequent inspection and/or maintenance intervals.

Deterioration of the spread footing concrete can result in breaking off the footing projections or spalling the sides. Severe deterioration may be caused by ice and debris pounding against the upstream side of the footing, water penetration resulting in corrosion of the reinforcing steel, or poor material in the footing. The area of the footing must not be reduced, as the load of the bridge must be distributed uniformly upon the material under the footing. The repair of the footing proceeds as follows:

Suggested Procedure

Footing Repair

1. Water must be kept clear of work area by means of diversion channels, cofferdams, sandbags, or sheet piling if required.
2. Move the traffic to the opposite side of the bridge.
3. Install shoring and jack the superstructure to remove the loading on the element, if necessary.
4. Chip away the deteriorated concrete until sound concrete is reached. Clean away all loose concrete with air blast or other means.
5. Install reinforcing bar, anchors and rods if they are to be used.
6. Construct forms that are adequate to restore the footing dimensions to the original size. It is common to extend the footing to cover a larger area and extend the sides downward if undermining has occurred.
7. Apply an appropriate epoxy compound (if owner allows)* or a neat cement paste for bonding just prior to pouring the new concrete into the forms.
8. Mix and pour the new concrete using a strong mix with low slump. Vibrate the concrete thoroughly to ensure a dense pour and a good bond.
9. When the new concrete has been cured for at least three days, or longer if specified, remove the forms.
10. Replace loading on footing and remove jacking and shoring, if necessary.
11. Remove any cofferdam and restore the stream channel to its proper course.

* Not all agencies authorize the use of bonding agents.

Most piles require little maintenance because the material into which they are driven protects them, and subsurface damage or deterioration is not common at lower depths where oxygen concentrations are lower. Where piles are exposed, whether by design or by scour, there are potential problems. These problems include scaling and spalling of concrete piles, corrosion of metal piles or decay in timber piles and buckling in all types, if the unsupported length becomes excessive.

It is not always safe to assume that a buried pile does not corrode. There have also been documented occurrences of corrosion of piles due to saturated soil and a high ground water table.

13.4.3 Jacketing of Concrete Substructure Elements

Jackets are the most common type of pile protection or repair. They are used for protection of all types of piles: concrete, steel, and timber. The jacket can be for protection from abrasion damage, repair of section loss, or both. If the jacket were for protection only, it would consist of a liner placed around the area to be protected with a cementitious grout or epoxy resin pumped into the annular opening between the existing concrete and the liner. If the jacket is intended to repair structural damage the liner will provide space for reinforcement and the space between the liner and old pile is filled with concrete. The liner (form) is often premolded

fiberglass; however, it could be steel or fabric. Old drainpipes have been used as jacket liners. Figure 13.29 shows a standard concrete pile jacket with steel reinforcing cage.

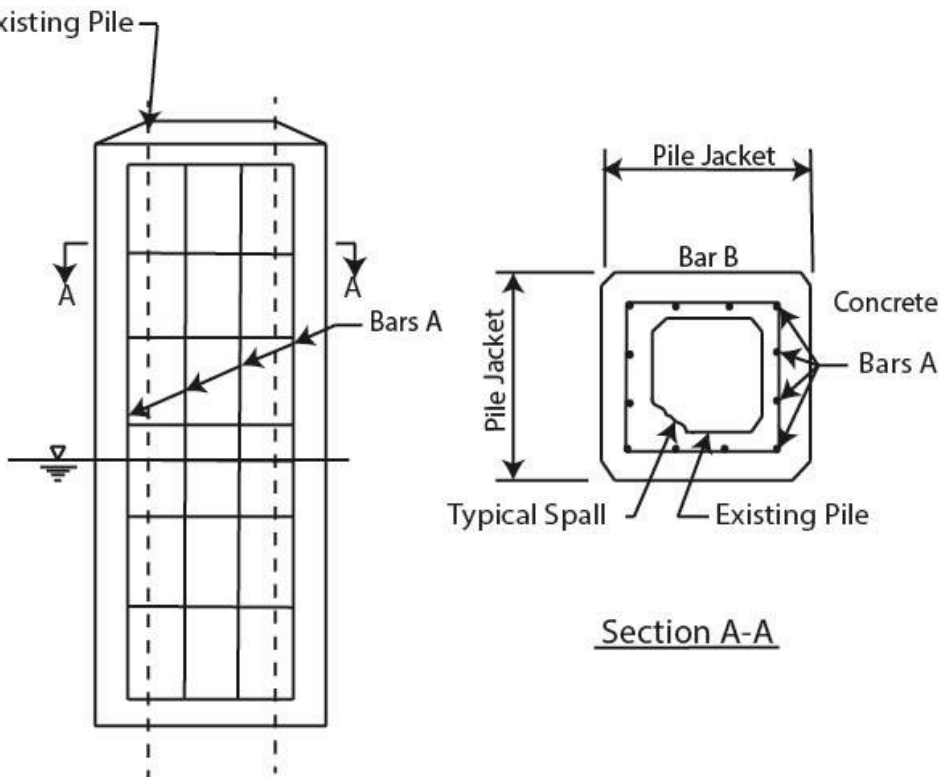


Figure 13.29 Concrete Pile Jacket with Steel Reinforcing Cage

Deteriorated reinforced concrete and prestressed concrete piles can be encased with a concrete jacket after all unsound concrete has been removed and the surface prepared as described previously.

When to Call the Engineer

Call the Engineer before any concrete removal of prestressed piles.

Encasement will compensate for the cross-sectional loss and strengthen the pile. Deteriorated concrete in a concrete pile should be removed until sound concrete is exposed. The reinforcing steel should then be cleaned of all rust and scale, and new concrete placed. Sufficient concrete should be removed so that new concrete is replaced to a minimum of 2 inches in depth.

Reinforcing steel cages or reinforcing wire is placed around the pile before forms are placed. The reinforcement is usually epoxy-coated for protection against corrosion. Stand-offs are placed on the reinforcement before they are drawn tight to the pile. Forms, either rigid or flexible, are then installed and sealed. Concrete is placed in the form either by tremie or dewatering the form if the concrete placement is below the waterline. After placing the concrete, the forms are either left in place permanently for further protection of the pile or removed when the concrete is cured.

Special requirements for the installation of concrete-filled pile jackets include qualified divers for underwater survey and repairs and a concrete pump for underwater placement.

Suggested Procedure

Concrete-Filled Pile Jacket Installation

1. Scrape surface of the pile clean, removing deteriorated concrete or wood.
2. Sandblasting may be used to clean the exposed reinforcement in concrete piles above the waterline. Splice with new reinforcement if required. Install steel mesh reinforcing cage around timber pile or concrete pile. Use spacers to keep the forming and reinforcement in proper position.
3. Place the forming jacket around pile and seal the bottom of form against pile surface.
4. Pump suitable concrete into form through opening at the top. Sulfate-resistant concrete should be used in salt-water locations.
5. Finish top portion of repaired area.

An example detail showing the installation of concrete-filled pile jackets is shown in Figure 13.30.

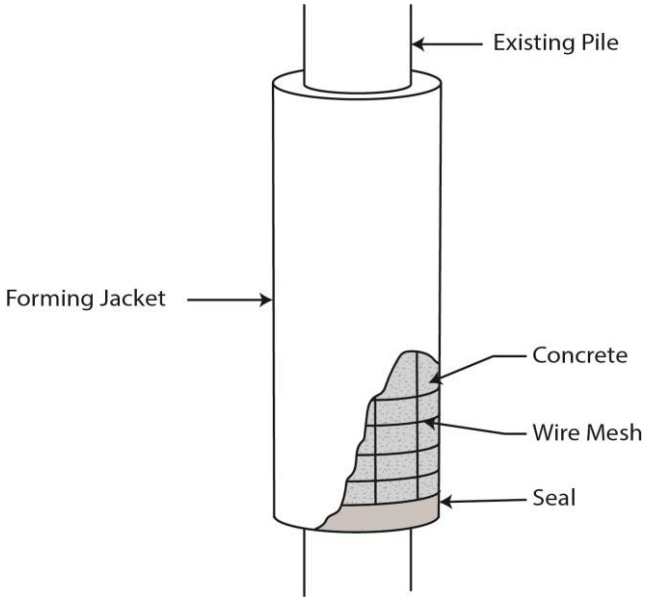


Figure 13.30 Concrete Pile Jacket Installation

Fiberglass forms can be used to construct pile jackets. The forms are light, can be easily installed, and create a watertight protective layer for the pile. Damage normally extends above and below the waterline. Deteriorated concrete is removed using high-pressure water jets. The jacket should extend approximately 2 feet beyond the damaged area at each end of the pile to account for any concrete segregation near the bottom or loose materials at the tops of the new concrete. Welded wire fabric or a reinforcement cage is wrapped around the repair area.

Fiberglass forms have a vertical seam, so that it can be fitted around neatly, top and bottom centering devices and a bottom seal are placed. The form is secured in place with bolted ends and tightened to ensure full enclosure. If the length of the repair exceeds the length of the form, the piles may be repaired in two lifts. If the damage extends below the mudline, trenches are dug at the bottom to extend the repair into the mud zone.

Underwater, exposed reinforcing steel in concrete piles can be protected against corrosion by attaching zinc anodes to them, as shown in Figure 13.31. If the pile is repaired and jacketed, the reinforcing steel that is embedded in sound but chloride contaminated concrete has potential for corrosion. This corrosion and the subsequent concrete deterioration may be undetected because of the presence of the jacket. Attaching zinc anodes to the exposed reinforcing steel in the cavity prior to repair and jacketing can mitigate or stop the ongoing corrosion of the reinforcing steel in jacketed piles. Commercially available zinc anodes that are embedded in cementitious materials may be used for this purpose. Manufacturer recommendations should be followed.

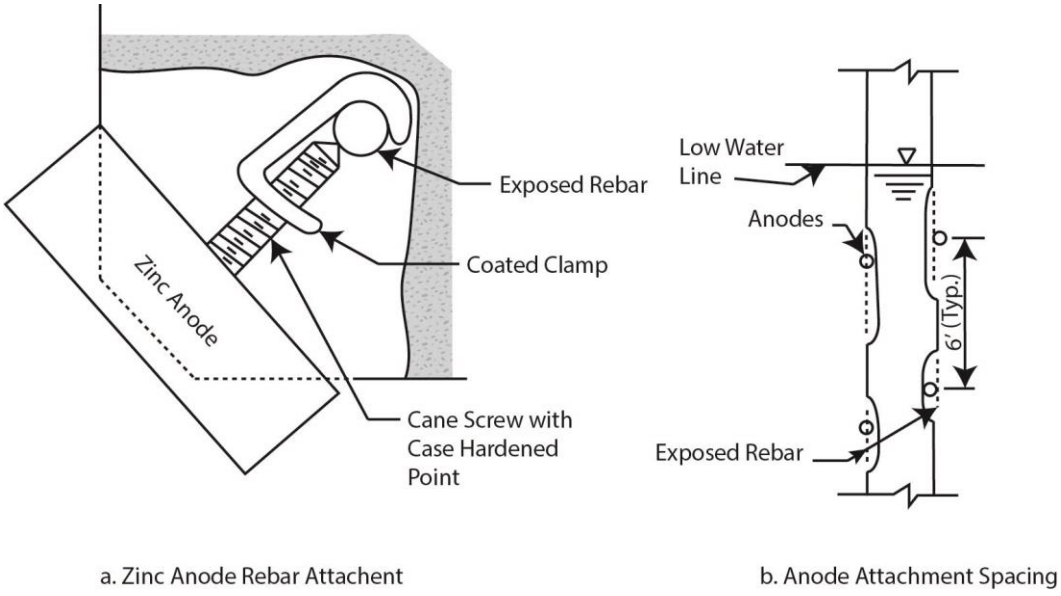


Figure 13.31 Galvanic Cathodic Protection of Concrete Piles (Underwater)

Filled shells are cast-in-place concrete piles. A metal shell is driven first, the mandrel withdrawn, and the shell is then filled with concrete. A problem that sometimes develops is corrosion of the shell due to deterioration of the concrete.

Suggested Procedure

Cast-In-Place Pile Repair

1. Erect a temporary shoring bent to assume the load when necessary.
2. Remove rust and scale.
3. Prime and paint steel.
4. Where both the shell and the concrete are damaged, remove the deteriorated portion of the shell and place a collar of sufficient strength and diameter placed around the pile.
5. Ensurethe collar is extended well above and below the affected area and pressure inject a high quality, low shrinkage concrete, epoxy mortar or cementitious grout inside the collar to fill voids.
6. Compact the material so voids are completely eliminated.

13.4.4 Shotcrete for Substructure Surface Repairs

Shotcrete can be used in the repair of substructure concrete. Shotcrete is a concrete or mortar pneumatically projected at high velocity onto a surface. Shotcrete repair is effective for the repair of bridge beams, caps, piers, abutments, wing walls and decks. Since forms are not generally used for shotcrete, it is particularly effective on the underside of a deck for an overhead patch where there is no possibility of using a form.

The advantages of shotcrete are:

- Superior bond
- Greater strength due to high density
- Low shrinkage
- Requires no formwork
- Ability to use in overhead repairs

The disadvantages of shotcrete are:

- Space required for application
- Skill required for application
- Appearance and aesthetics
- High cost, particularly for small quantities

When the shotcrete method is used, no bonding agent is typically necessary. When the depth of the patch exceeds 3 inches, hook or expansion anchors should be used to secure to the existing concrete. When shotcrete patches are over traffic or public spaces, an anchorage system should be used regardless of patch depth. A typical anchorage system may consist of welded wire fabric (plain, galvanized, or epoxy coated) on 18-inch centers and 2-inch by 2-inch wire fabric hooked and wired to the anchors. This anchoring system may be repeated for every 3 inches of depth of the shotcrete applied 2-1/2 inches beyond that layer of steel, as shown in Figure 13.32. This provides an excellent anchor for the new concrete or shotcrete; if removal

stopped at the plane of reinforcement, a cleavage plane is apt to develop at the interface between the old and the new concrete. Rust and other harmful an materials should be removed from the reinforcing steel. Sandblasting should be used in those cracks where reinforcing steel is exposed because it cleans the concrete as well as the rust off the reinforcing steel.

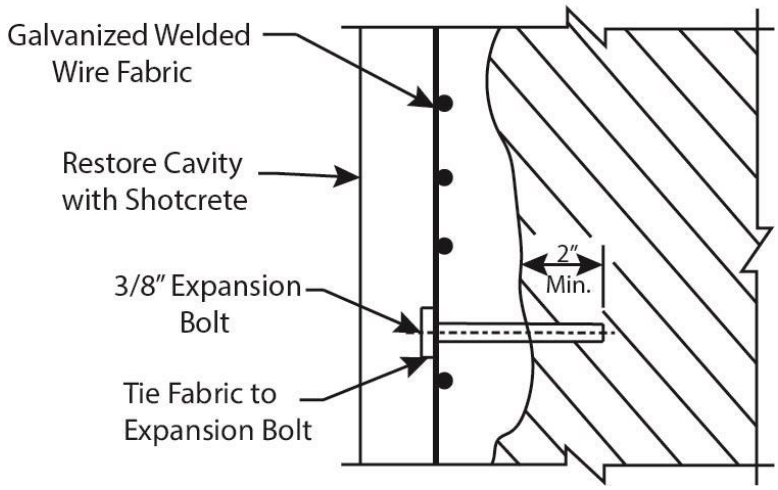


Figure 13.32 Welded Fire Fabric Anchorage for Shotcrete

Shotcrete Considerations:

- Wet or Dry
- Surface Preparation
- Skilled Nozzleman
- Mix Design
- Alignment Control
- Finishing and Aesthetics
- Inclusion of Fibers

Shotcrete Surface Preparation:

- Remove Loose or Unsound Material
- Transition Changes in Thickness
- Abrasive (or Hydro) Blast Surface
- Avoid Feathered Edges
- Pre-wet Surface

Shotcrete Mix Design Considerations:

- Prepackaged Mix
- Cement/Aggregate (1/3-1/5)
- Low Water-Cement Ratio
- Superplasticizers (Wet Mix)
- Air-Entrainment (9-12 percent Wet Mix)
- 7-15 percent Silica Fume
- Accelerators

Shotcrete Application Considerations:

- Assure Uniform Flow of Material
- Apply Thin Bond Coat First
- Maintain 90 Degree Nozzle Spray Angle
- Vary Angle Around Rebars
- Apply in Thick Layers
- Remove Rebound Ahead
- Cure to Avoid Cracking

13.4.5 Crack Repairs

Sealing cracks in substructure elements typically involves vertical or horizontal application of the material. However, before a method is identified for repairing a crack, it must be determined if the crack is working (i.e., active or growing) or not. If the crack is working it should be filled with flexible material. If it is passive a bonding material such as an epoxy can be injected into the crack. It is also necessary to determine if the crack is full depth. This may be difficult if the crack is in an abutment.

Narrow cracks that are dormant may be effectively sealed by epoxy injection. The procedure can be applied to both horizontal and vertical surfaces, though low viscosity polymers can be applied by the gravity-feeding approach for horizontal surfaces. Cracks as narrow as 0.002 inches can be sealed and bonded by the injection of epoxy. The procedure has potential to provide structural repair (increase stiffness and strength) in addition to sealing the crack. Chapter 5 has procedures for crack repair.

A footing may crack transversely due to uneven settlement of the pier or abutment. This will often be accompanied by a crack continuing up through the pier or abutment. It is advisable to seal the crack to prevent further intrusion of silt, debris, and water, which will attack the reinforcing steel. If the crack is moving it should be filled with a flexible material; otherwise, it will crack again. If the crack is not moving it can be bonded back together.

Epoxy Injection

The most effective method of repairing substructure vertical cracks is epoxy injection. An example of the installation ports is shown in Figure 13.33. The crack should be sealed full length (on both sides if it is a wall) before injecting the epoxy. Application of epoxy resin to seal the cracks is shown in Figure 13.34.

Figure 13.33 Installation of Epoxy Injection Ports (Courtesy of MDOT)

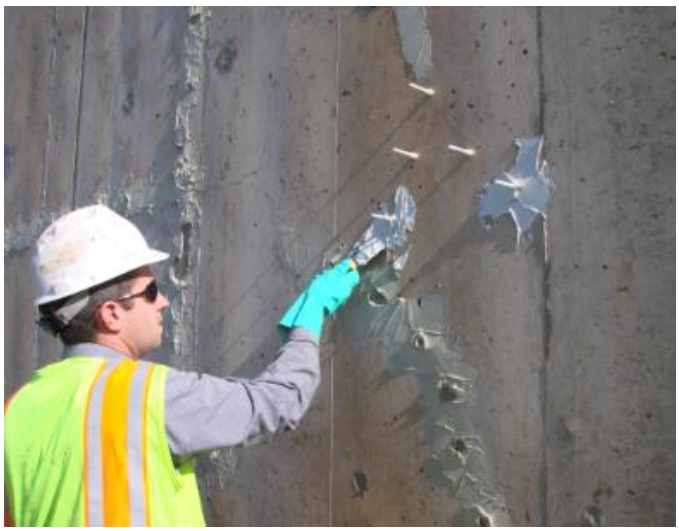


Figure 13.34 Application of Epoxy Resin over Cracks Prior to Injecting (Courtesy of MDOT)

To get maximum penetration of the epoxy filler, the first injection is made at the bottom of the crack. Starting at the bottom and moving up in gradual increments toward the top increases the pressure needed to apply the epoxy and should result in greater crack-filling penetration.

The injection progresses from port to port, normally starting at the lowest point, and continuing until epoxy is extruded from the next port. The distance between the ports should not exceed the expected penetration depth. A handgun or pressure pot can be used, but various types of machines are available that assure the proper proportioning, mixing, and temperature of the two part epoxy and the proper injection pressure. This technique may also be used to fill isolated voids or delamination in concrete. In this case, injection pressure must not be too high.

Chapter 5 covers epoxy injection repair procedures in greater detail.

13.4.6 Mechanisms that Move Substructures

To better understand how to repair concrete substructures, it is important to understand why the structures settle. The mechanisms that drive the movement of the substructures can be related to:

Slope failure (embankment slides): These are shear failures manifested as lateral movements of hillsides, cut slopes, or embankments. Footing or embankment loads imposing shear stresses greater than the soil shear strength are common causes of slides. Slides usually occur during wet conditions which reduces the soil shear strength.

Bearing failures: Bearing failures are settlements or rotations of footings due to a shear failure in the underlying soil. When bearing or slope failures take place on an older structure, it usually indicates a change in subsurface conditions. This may endanger the security of nearby structures and foundations.

Consolidation: Serious settlement can result from consolidation action in cohesive soils. Settlement of bridge foundations may be caused by changes in the groundwater conditions from drawdown, the placement of additional embankments near the structure, or increases in the height of existing embankments.

Seepage: The flow of water from a point of higher elevation through the soil to a lower point is seepage. Seepage force acts on the soil through which the water is passing. Seepage results in lateral movement of retaining walls by:

- An increase in weight (and lateral pressure) of the backfill because of full or partial saturation.
- A reduction in resistance provided by the soil in front of the structure.

Water table variations: Large cyclic variations in the elevation of the water table in loose granular soils may lead to a compaction of the upper strata. Changes in the water table may also change the characteristics of the soil that supports the foundation. Changes in soil characteristics may, in turn, result in the lateral movement or the settlement of the foundation.

Frost action: Frost heave in soil is caused by the growth of ice lenses between the soil particles. Footings located above the frost line may suffer from the effects of frost heave and a loss in bearing capacity due to the subsequent softening of the soil. The vertical elements on light trestle bents may also be lifted by frost and ice actions.

Expansive soils: Some clays, when wet, absorb water and expand, placing large horizontal pressures on any wall retaining such soil. Structures founded on expansive clay may also experience vertical soil movements (reverse settlement).

Ice: Ice can cause lateral movement in two ways. Where fine-grained backfill is used in retaining structures and the water table is above the frost line, the expansion of freezing water will exert a very large force against a wall. The piers of river bridges are also subject to tremendous lateral loads when an ice jam occurs at the bridge.

Thermal forces: On structures without expansion bearings, or where the expansion bearings fail to operate, thermal forces may tip the substructure units. Pavement thrust is another force that will have the same effect.

Drag forces: Additional embankment loads or very slow consolidation of a subsurface compressible stratum will exert vertical drag forces on the bearing piles which are driven through such material. This may cause yielding or failure of the piles.

All piles may develop weaknesses from deterioration, insect attacks, and construction defects. These weaknesses may lead to foundation settlement:

- Timber, steel, and concrete piles are subject to loss of section because of decay, rusting, and deterioration.
- Timber piles are vulnerable to marine borers and shipworms.
- Construction defects include overdriven piles, under-driven piles, and failure to fill pile shells completely with concrete or imperfect casings of a cast-in-place pile.

Settlement will probably be gradual in improperly driven piles or in piles with weak or voided concrete. Piles suffering severe loss of section due to rust, spalls, chemical action, or insect infestation may fail suddenly under heavy load.

Scour and erosion: Scour can cause extensive settlement and/or structural failure. Since water will carry off particles of soil in suspension, a considerable hole can be formed around piers or other similar structural objects. This condition results in a greater turbulence of water and an increased size of soil particles that can be displaced. Erosion of embankments due to improper drainage can also lead to approach and abutment settlements. A common occurrence is erosion loss of the toe of an embankment which is problematic because slope stability is reduced. The reduction in slope stability causes either surface failures or deeper subsurface slides.

Earth or rock embankments (stockpiles): Post-construction placement of embankments may cause instability since it will produce greater loads than were included in the original design.

13.4.7 Settlement Repairs

All foundations undergo some settlement. However, very small foundation movements have no effect. Simple span structures, and those with joints, will tolerate even moderate differential displacements with little difficulty other than minor cracking and the binding of end dams. Movements of large magnitudes, especially when differential, cause distress in nearly all structures. Tremie Concrete Scour Repair. Schematic with arrows indicating water level, one half inch diameter forming struts, welded wire mesh reinforcing, 12 gauge metal forming left in place, tremie concrete enclosement, heavy stone riprap to control future undermining. Large movements will cause deck joints to jam; slabs to crack; bearings to shift; substructures to crack, rotate, or slide; and superstructures to crack, buckle, and possibly, even to collapse. The larger the settlement to be accommodated within a given distance, the more structural damage can be anticipated.

13.4.7.1 Sensitivity of Bridges to Settlement

The types or settlement are categorized as:

Uniform settlement. A uniform settlement of all the foundations of a bridge will have little effect upon the structure. Small single-span bridges have experienced settlements of roughly 1 foot with no sign of appreciable distress.

Differential settlement. Differential settlement can produce serious distress in any bridge.

Where the differential settlement occurs between different substructure units, the magnitude of the damage depends on the bridge type and span length. For example, a rigid frame arch type structure is so stiff any significant differential movement could produce large cracks. However, if the structure is tall and flexible and with superstructure joints over the piers it should be able to handle some settlement without severe distress in the concrete. Should a differential settlement take place beneath the footings of the same substructure, damage can

vary from an opening of the vertical expansion joints between the wing wall and the abutment to severe tipping and cracking of walls or other members. Types of differential settlement include simple span and continuous span:

- *Simple span differential settlement.* As mentioned, movements usually do not affect the strength of a simple span structure unless they are quite large. There are usually enough joints to permit the movements without major damage to the basic integrity of the structure. At most, some finger joints or bearing may require resetting, or beam supports may need shimming. However, pile bent or trestle bridges are very vulnerable since a large settlement or movement of a bent could cause the superstructure to fall off a narrow bridge seat, leading to the loss of the bridge spans.
- *Continuous span differential settlement.* Differential movements seriously affect a continuous bridge, since such movements at supports will redistribute the loads, possibly causing large overstresses. These bridges are very likely to be damaged if subjected displacements that are greater in magnitude or different in direction, from those considered in the original designs.

Following are the most common causes of foundation movement and are classified by the type of movement:

- Lateral movements - Earth-retaining structures, such as abutments, wingwalls and retaining walls, are susceptible to lateral movements from either bending or sliding although piers sometimes also undergo such displacements. Both shallow and deep foundations are subject to lateral movement.
- Settlement (vertical movement) - Any type of substructure not founded on solid rock may be subject to settlement. Again, both shallow and deep foundations may settle. Settlement can be caused by
    - Erosion of soil
    - Erosion of weathered or soluble rock
    - Inadequate bearing capacity of soil
- Rotation (tipping) - Rotation can be considered to be the result of unsymmetrical settlements or lateral movements. It will be discussed under the movement that is typical of the various substructures.

The common causes of foundation movement leading to foundation failures are shown in Figure 13.35.

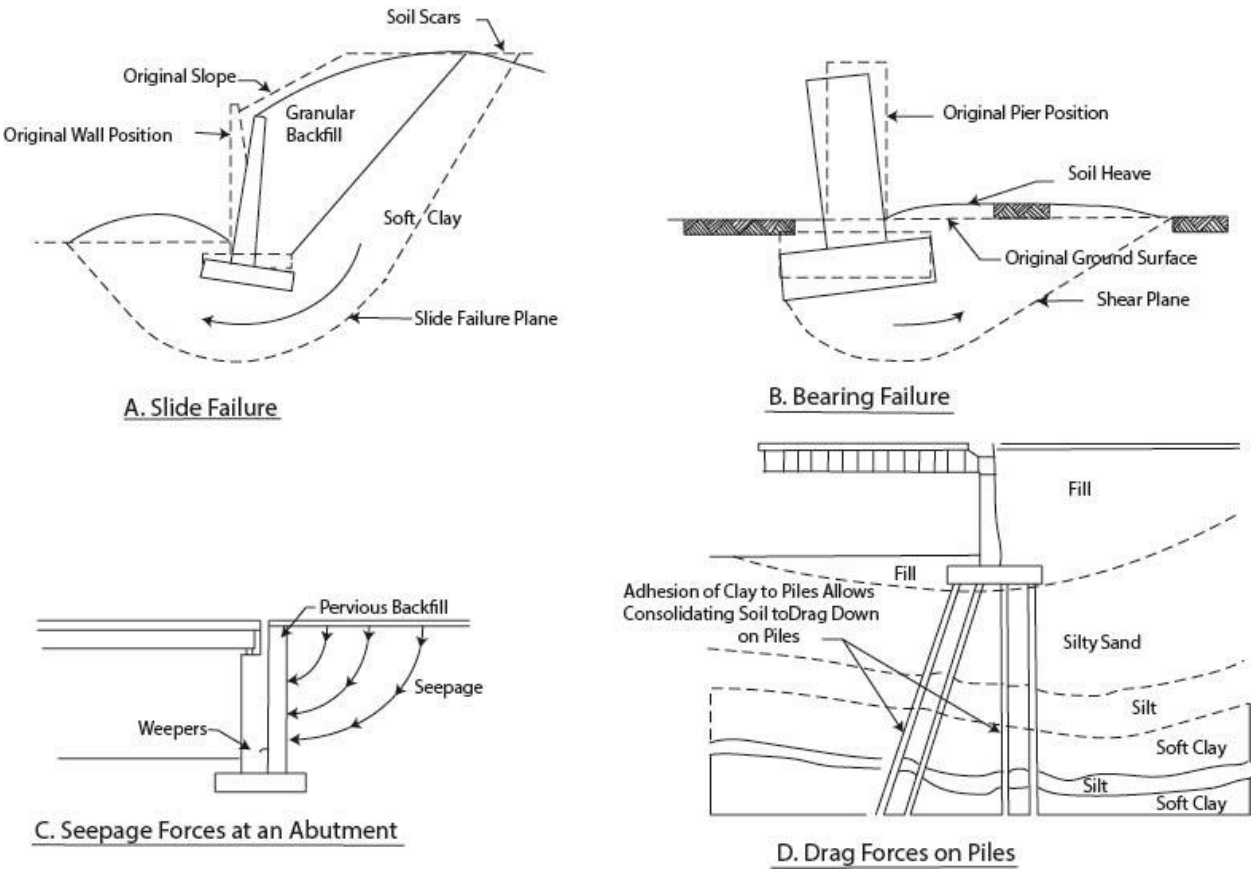


Figure 13.35 Common Causes of Foundation Failure

13.4.7.2 Repairs to Mitigate Underwater Differential Settlement

Channel scour can cause undermining of footings resulting in differential settlement of pier. Corrective work includes the following:

Preliminary planning includes:

- Will shoring be required if the substructure will be lowered to an elevation below existing condition.
- Thoroughly evaluate condition of foundation under pier footing.
- Establish new elevation with footings leveled and stabilized.
- Design new bearing pedestals to bring superstructure to proper grade.

When to Call the Engineer

Call the Engineer when planning to correct differential settlement.

Resource requirements are:

- Diving equipment
- Work boat

- Concrete drilling equipment
- Steel dowels
- Concrete pumping equipment
- High strength pretailored grout bags
- Reinforcing steel
- Jacks for bringing the superstructure to grade

A procedure for correcting underwater differential settlement follows:

Suggested Procedure

Correction of Underwater Differential Settlement

1. Remove traffic.
2. Install turbidity curtain and other devices to maintain water quality. Alternately, a cofferdam could be used, and repairs constructed in the dry.
3. Install concrete leveling sill to ensure pier stability during excavation. The sill consists of an appropriately positioned concrete grout bag extending the entire width of the pier.
4. Remove protruding boulders under footing.
5. Excavate to level footing using high-pressure water jets subject to environmental water quality requirements.
6. Install grout bags and fill with pressurized concrete to mold to and completely fill the cavity under pier. Engineered design of scour countermeasures installed below the depth of total scour is required.
7. Place grout bags around periphery of the pier to increase footing size and depth, thereby reducing further potential for undermining.
8. Install horizontal and vertical reinforcement through the grout bags.
9. Drill and grout dowels on 3-foot centers into existing stem and footing to anchor new work to old.

Figure 13.36 shows a procedure for correcting differential settlement.

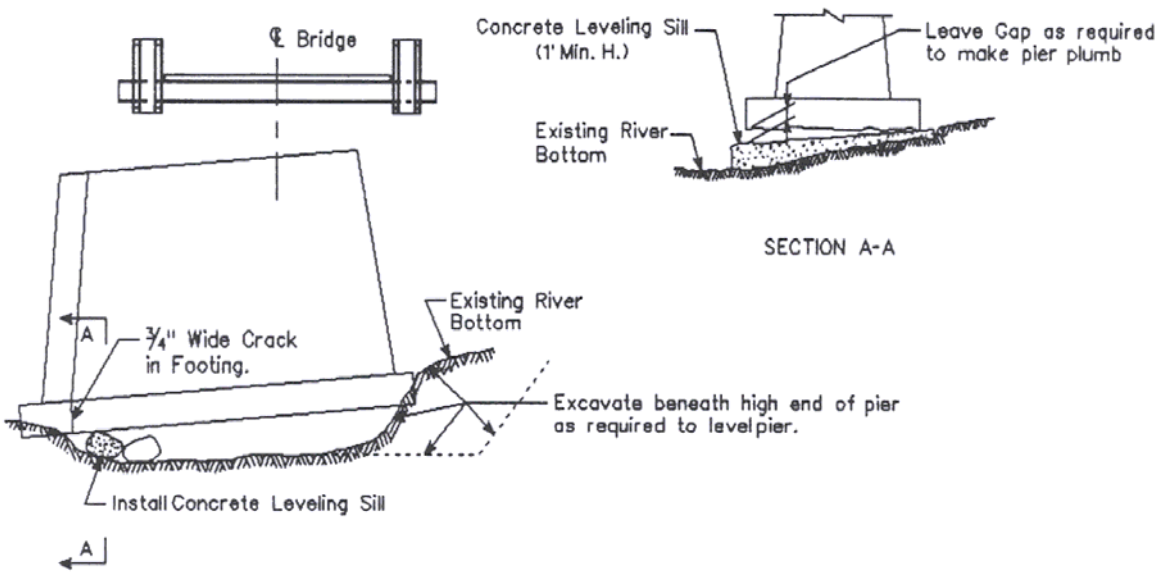


Figure 13.36 Pier Settlement Stabilization

When to Call the Engineer

Call the Engineer for any pier settlement stabilization repair.

Where serious settlement is expected (either due to live loads or seismic loads), a soils and/or structural engineer should be consulted prior to taking measures to prevent the damage to the abutment or pier. A footing that is resting upon piles may require additional piles to stabilize it.

Suggested Procedure

Pier Settlement Stabilization Repair

1. Excavate around the footing.
2. Remove concrete to expose the edges of lower mat of reinforcing steel.
3. Clean the existing concrete and reinforcing steel.
4. Drive all additional new piles (not required for spread footing).
5. Form the new extended footing.
6. Lap the new reinforcement to the existing exposed bars. If lapping is not feasible, drill and grout the new reinforcement to the existing concrete.
7. Place new top mat bars and the reinforcing cage.
8. Place and cure the concrete in the extended footing.
9. Backfill over the footing.

Figure 13.37 shows how additional piles could be added to the footing and the footing widened and thickened.

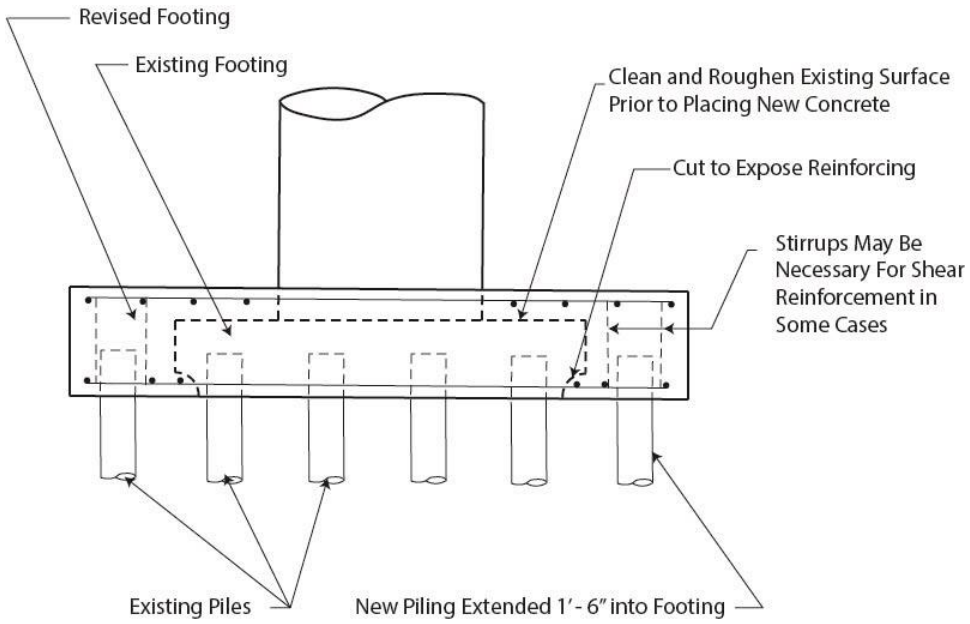

Revised Footing
Existing Footing
Clean and Roughen Existing Surface Prior to Placing New Concrete
Cut to Expose Reinforcing
Stirrups May Be Necessary For Shear Reinforcement in Some Cases
Existing Piles
New Piling Extended 1′ - 6″ into Footing

Figure 13.37 Increasing Live Load Capacity of Existing Footing

13.4.8 Undermining Repairs

When making repairs to bridge undermining the following should be considered:

- Has the undermining been previously repaired with an engineered solution to the scour problem?
- Will the repair be temporary or permanent? A temporary repair can be as simple as replacement of the eroded material back into the scour holes.
- Should the repair be engineered to ensure it extends to below the scour depth?

Most repairs for scour around bridge piers and abutments will involve the placement of both riprap and concrete. There are several special considerations that apply to the placement of these materials around bridge substructure units.

Placement of concrete to repair scour or undermining of substructure units usually requires either dewatering or underwater placement of concrete. In either case proper placement, good forms, and skilled personnel are essential. Dewatering may be accomplished by constructing cofferdams, or if environmental regulations permit, diversion of the flow away from the repair area. The primary disadvantage of dewatering with cofferdams is the clearance required for driving sheet piling. General precautions for concrete placement under wet or dry conditions include:

- An analysis should be performed to ensure that pile support would not be overloaded.
- Forms should be prevented from moving and should not be removed prematurely. Prebagged concrete or concrete pumped into fabric forms are often used in place of conventional forms.
- Concrete should not be placed in running water or be allowed to drop through water as this could wash the cement out of the concrete.

An example of an undermining repair is shown in Figure 13.38, showing (a) Existing conditions before the scour remediation work, (b) Access and mobilization to place water control, (c) Sandbag water control in place, (d) Dewatering operations, (e) Exposed footing showing undermining, and (f) Placement of concrete to fill voids under footing. The completed repair is shown in Figure 13.39.

Figure 13.38 Undermining Repair Sequence

Figure 13.39 Final Condition with Streambed Restored

Underwater placement can be accomplished by underwater tremie, pumping, bagged concrete, and prepacked concrete. Descriptions and special considerations for each of these methods are briefly discussed in the following paragraphs.

A tremie is a tube with a hopper at one end and a discharge gate at the other. It is used to place concrete under water by gravity flow. The discharge gate end is closed until the tremie is filled with concrete and lowered to the point where the concrete is to be placed. Once the pour begins, the discharge end must be kept submerged in the newly placed concrete and the tremie hopper must be kept filled with concrete. If this is not done, the pipe will fill with water. Multiple tremies are needed to reduce lateral travel and when reinforcing steel restricts the movement of the tube.

Placement of concrete by pumping is similar to tremie placement. Since, the concrete is pumped rather than gravity fed it is somewhat easier to control the discharge. A manifold can connect multiple hoses.

Concrete can be placed underwater in bags made of a porous material such as burlap. The bags are partially filled with concrete and then placed by workmen in shallow water or by divers in deeper water. An example of grout bags used to prevent undermining is shown in Figure 13.40. Bags may be filled with dry concrete and wetted during placement. Concrete may also be pumped into fabric forms specially sized to fill a void under the footing.

Prepacked concrete involves filling the forms with coarse aggregate and pumping in a grout. The grout displaces the water and fills the voids in the aggregate. Special equipment and techniques are required for this method.

Figure 13.40 Grout Bags Used to Prevent Undermining

13.4.8.1 Tremie Concrete Repairs

Undermining of footings that are not on piles can result in reduction of load capacity, settlement, tipping, or failure of the structure. Undermining of footings that are supported by piles can also be a serious problem. Exposure of the piling reduces the pile load capacity and exposes the piling to increased corrosion and abrasion. The load-carrying capacity of the bridge is also reduced because the footing has less support. Repair of undermined pier foundations is accomplished by the use of a tremie concrete encasement to fill the void and installation of riprap to prevent future undermining. An example of a tremie concrete scour repair is shown in Figure 13.41.

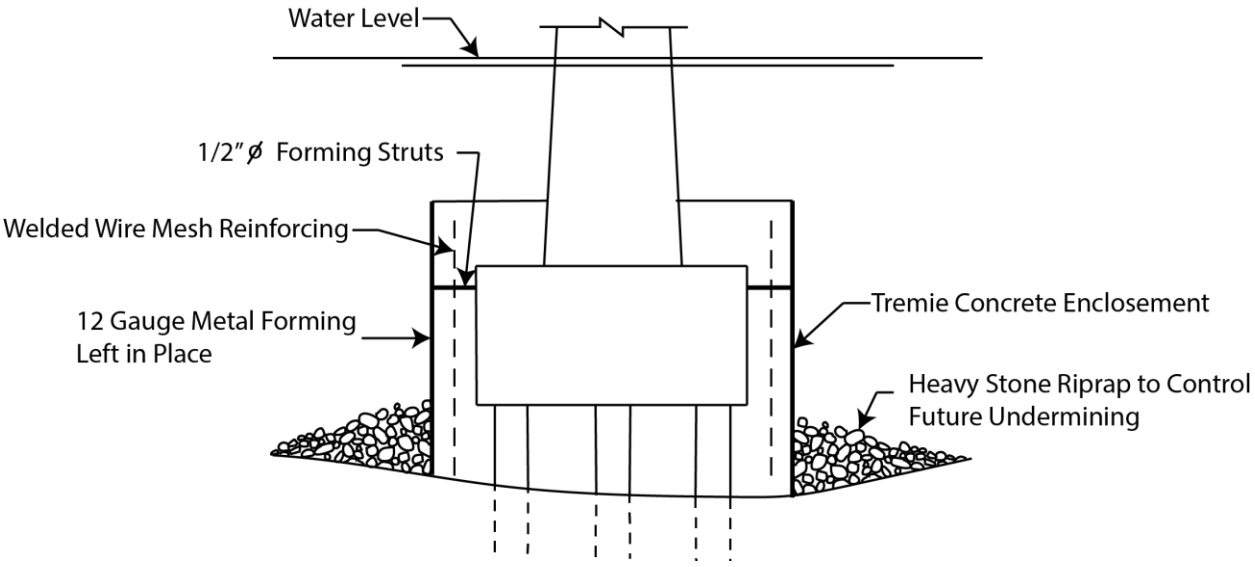


Figure 13.41 Tremie Concrete Scour Repair

Planning should include the following:

- Analyze the flow characteristics to determine the cause of the problem.
- Consider of the additional weight of the concrete encasement on the piling and soil foundation.
- Fabricate metal forming and reinforcement.

When to Call the Engineer

Call the Engineer when planning any tremie concrete scour repair.

The following will be needed:

- Pile driving equipment if cofferdams are used
- Pumping or tremie concreting equipment
- Crane or other lifting equipment

Suggested Procedure

Tremie Concrete Scour Repair

1. Clean all exposed concrete surfaces of aquatic growth and loose or deteriorated concrete to ensure a good bond.
2. Place welded wire mesh around the footing. Rebars may sometimes be used to withstand shear load if the encasement significantly spreads the footing.
3. Set forms and pour tremie concrete.
4. Leave metal forms in place.

An alternate to the preceding procedure is to substitute dry-mixed concrete riprap in bags for the steel forms, as shown in Figure 13.42 below.

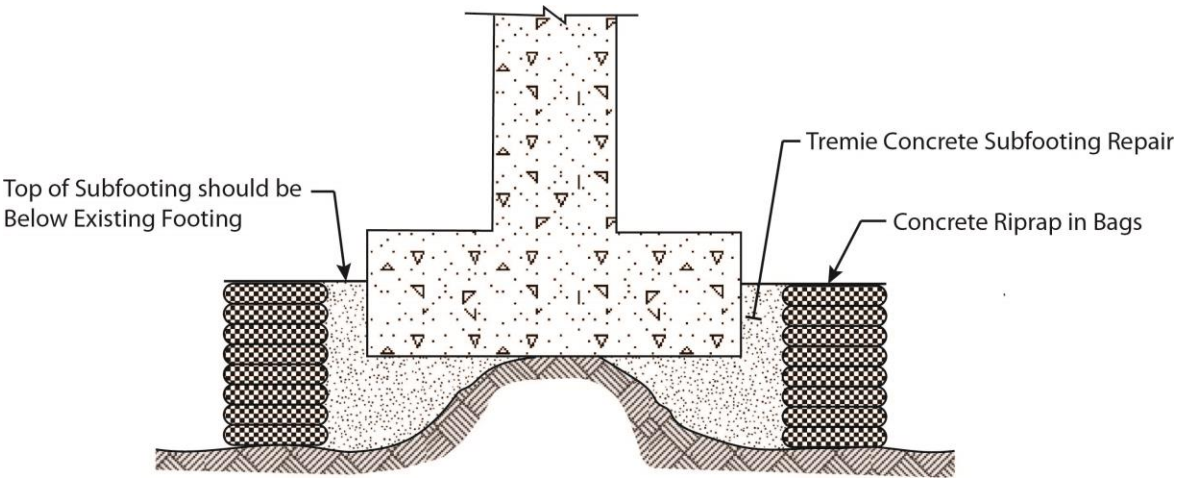


Figure 13.42 Bagged Riprap Scour Repair

13.4.8.2 Mortar Filled Tube Scour Repair

For small voids under the footings, a technique using fabric tubes filled with mortar and grout injection pipes is an alternative that can prove to be cost effective. See Figure 13.43 for an example of tremie repair using fabric tubes.

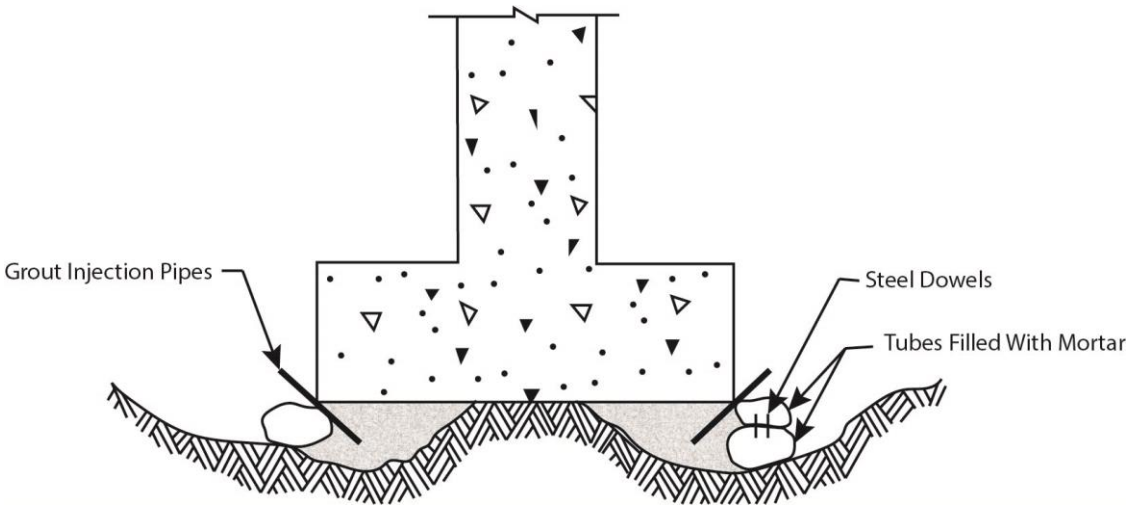


Figure 13.43 Mortar Filled Tube Scour Repair

13.4.8.3 Eroded Scour Repair

The undermining of abutment footings results from causes similar to the undermining of pier footings. The consequences are also similar with the additional hazard that the abutment is likely to be asymmetrically, not evenly, loaded due to the approach fill pushing against it. These forces may tend to tip the abutment toward the streambed. The magnitude of asymmetric loads, as well as deadmen that may have been provided to resist the loads, should be ascertained in evaluating safe conditions during repair. See Figure 13.44 for an example of eroded scour repair.

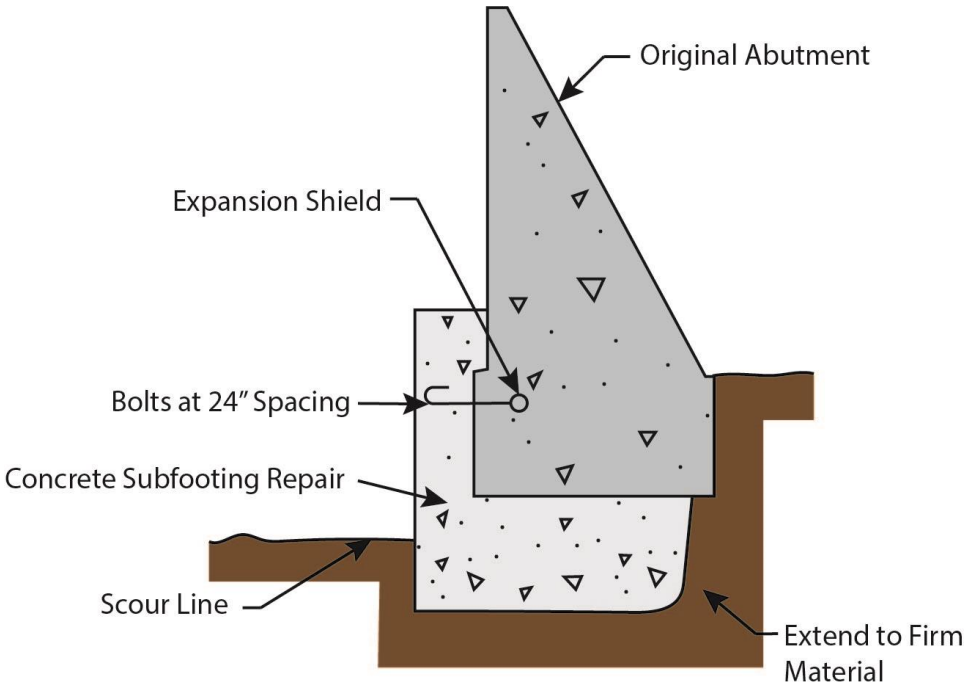


Figure 13.44 Eroded Scour Repair

Planning for the work must include the following:

- Analyze the flow characteristics to determine the cause of the problem.
- Evaluate safety problems during repair due to potential instability of the abutment.
- Analyze the hydraulic effect of stone riprap on the flow through the opening under the bridge.

When to Call the Engineer

Call the Engineer when planning any eroded scour repair.

The following equipment is usually needed:

- Equipment for placing riprap
- Pile driving equipment if needed to form the new footing
- Crane or other lifting equipment
- Concrete drilling equipment
- Concrete pump and tremie equipment

Suggested Procedure

Eroded Scour Repair

1. Remove loose material and silt from the foundation area.
2. Clean exposed concrete surfaces that will interface with subfooting.
3. Drill holes on 2-foot centers in existing concrete and install anchors.
4. Erect forms for the face of the subfooting.
5. Place concrete for subfooting using mix that is sufficiently plastic to fill all voids.
6. After the concrete has reached design strength, remove the forms and add stone riprap. Hint: See what size rock is not getting washed downstream. Use riprap equal to this size or larger.

13.4.8.4 Streambed Repair

The effectiveness of rebuilding a streambed depends on whether the scour problem is isolated in an area around the bridge or whether the streambed has been generally lowered. When the streambed has been generally lowered, it is best to extend the concrete footings below the scour line. This takes advantage of the deeper channel to reduce flow velocities. This procedure also aids in minimizing the construction of the channel by placement of riprap. Maintaining the maximum possible depth and width of a channel aids in reducing flow velocities which in turn reduces the scour action and the probability that any riprap used will be undermined or dislodged. An example of streambed paving repair is shown in Figure 13.45.

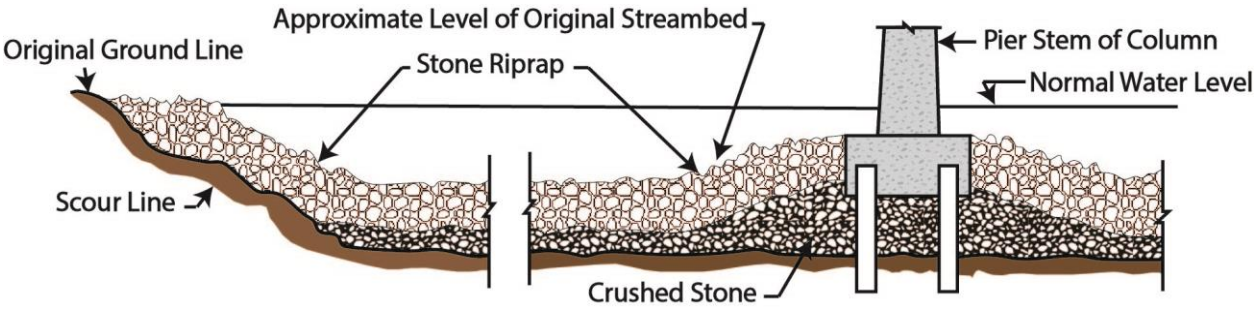


Figure 13.45 Streambed Paving Repair

The planning should include the following:

- Determine whether the streambed is generally lowered.
- Determine whether the extension of concrete footings below the scour line is feasible.
- Determine the velocity of flow and the size of riprap and crushed stone necessary to resist the scour.

When to Call the Engineer

Call the Engineer when planning any streambed paving repair.

The following material and equipment are required:

- Riprap, stone, or crushed stone
- Equipment such as front end loaders, excavators, or cranes to place the material

Suggested Procedure

Streambed Paving Repair

1. Place crushed stone in the channel to an elevation approximately 2 feet below the proposed top of the stream bed taking care to fill voids below the footings.
2. Add a layer of appropriately sized riprap for the stream over the crushed stone. Engineer should be contacted to determine the appropriate size and weight of riprap. The riprap should extend above the high water line at both sides of the channel.

13.4.8.5 Riprap Placement

The following factors should be considered when placing riprap around piers or abutments:

- Care must be exercised when dumping riprap around existing structural units. The large stones required can easily chip or break the concrete elements. Placement of riprap around piers must often be done from a barge.
- Placement should be made in even lifts to avoid unbalanced loads on footings.
- An analysis should be made to ensure that footings or pile supports would not be overloaded.
- The stone must be of suitable size for the anticipated flow conditions.

Riprap should not extend above the original streambed. Riprap that extends above the original streambed will act as an obstruction. The turbulence resulting from riprap improperly placed around a pier can cause localized scour at other piers or at the abutments. See Chapter 15 – Channel and Waterway for more details.

Planning must include the following:

- Evaluate the condition of the structure and the foundation to determine whether it is repairable and whether the pier can be stabilized in its present location.
- Evaluate the need for temporary supports until repairs are complete when traffic must continue to use the bridge.
- Design the stem enlargement and fabricated reinforcement.

13.4.9 Miscellaneous Abutment Repairs

Abutments vary in type and design and each has special considerations. Integral and to a lesser extent, semi-integral abutment structures have performed successfully for a number of years. This substructure type is defined in Chapter 3. For abutments on steel piles, deterioration at the soil line is a major concern. Wingwalls and Mechanically Stabilized Earth (MSE) can become unstable when steel ties to the wall face deteriorate and earth pressures behind the wall force

it to rotate. The same repairs for steel substructures can be used in the repair of integral or semi-integral abutments constructed with steel sheeting.

13.4.9.1 Repairs to Mitigate Lateral Movement in Abutments

The force of earth and stone in the bridge approach behind the bridge abutment tends to push the abutment forward and may tend to rotate (tip over) the abutment. These forces may exceed the resistance of the abutment if the fill behind the abutment is unstable or the abutment is not adequately anchored. An example of such as situation is the mounding of material in the median to prevent errant vehicles from traversing down the abutment slope.

Deadmen are used to provide a counter force to shoving or overturning pressures on the substructure. Deadmen are heavy masses (weights), usually concrete blocks, attached to the abutment with long steel rod. The deadmen are located in stable earth well behind the abutment to provide an anchor that prevents overturning of the abutment. See Figure 13.46 for a schematic of a deadman anchor installation.

Planning includes:

- Calculations to determine the magnitude of the forces to be resisted by the deadman
- Determining the required size of deadman, size of restraining rod and whether piles are required.

The resources required will include:

- Excavation equipment
- Concrete
- Miscellaneous hand tools
- Light lifting equipment
- Drills

When to Call the Engineer

Call the Engineer when planning to install deadmen for abutment stabilization.

Suggested Procedure

Deadman Installation

1. Excavate the area where the deadmen are to be placed and provide a trench for the restraining rods.
2. Drive piles for the deadmen, if required.
3. Place formwork and concrete for the deadmen. Note that the side of the deadmen facing the abutment should be cast without forms. All forms may be eliminated if the condition of the excavation permits.
4. Drill through the abutment stem, the portion below the seat, and place the restraining rods. Wrap and cast with tar or provide other means to protect rods from corrosion.
5. Bolt the restraining rods at the deadmen.
6. Place the waler beams and tighten the rods (see Figure 13.46).
7. Grout holes in backwall. Back fill.

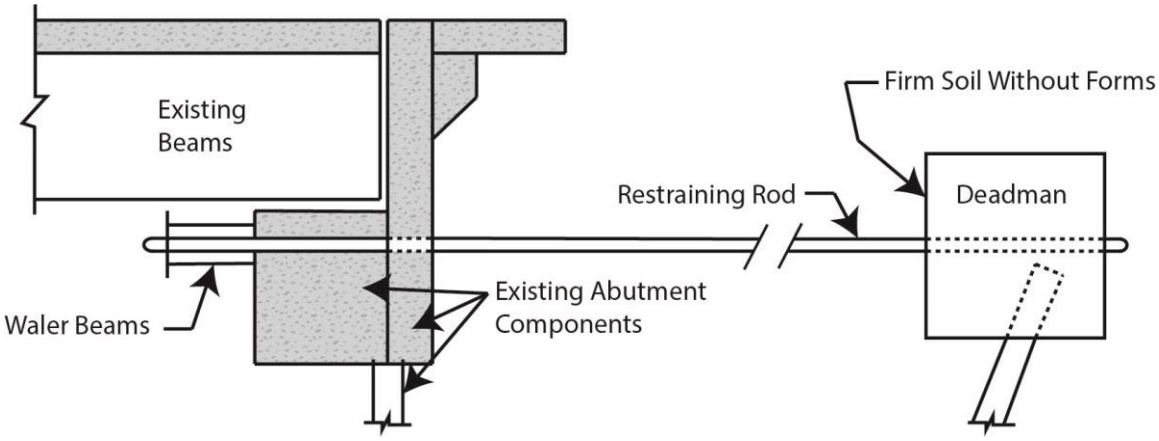


Figure 13.46 Installation of a Deadman Anchor

13.4.9.2 Bridge Seat Repair

If spalling and deterioration of the bridge seat concrete underneath a bearing is found to be severe enough and failure of the beam seat is imminent, repairs should be performed in these areas. The following is a suggested procedure for repairing spalled concrete bridge seats.

Suggested Procedure

Repair of Spalled Bridge Seat Concrete

1. Install traffic control devices (if necessary) and secure the work zone.
2. Jack the existing superstructure, if necessary, depending on the extent of the concrete deterioration. (See Section 12.3.2 Jacking of the Superstructure).
3. Remove the existing bearing devices.
4. The limits of the repairs shall be sawcut (or other) along neat lines to a depth of 1 inch where practical to produce a clean edge (see Figure 13.24). Remove deteriorated and unsound concrete. If removal exposes reinforcing steel, if over 50 percent of any bar is exposed, removal shall extend an additional 1 inch beyond the bar. Missing or deteriorated reinforcing steel shall be replaced.
5. Once removal of concrete is performed, if there is exposed reinforcing steel it shall be cleaned by mechanical cleaning and high pressure washing with water that contains no detergents or bond inhibiting chemicals. Where reinforcing steel has corrosion, it shall be sandblasted to a white metal finish. If it is found that the existing concrete is unreinforced, dowels or concrete anchors may be required to ensure the new concrete bonds and does not "pop out" of existing concrete.
6. Where new concrete is placed in contact with existing concrete, owner specifications shall be followed. If no specification exists, epoxy bonding compound shall be used in accordance with the manufacturer's instructions and applied to the existing concrete prior to placement of repair material.
7. Place new concrete material.
8. Allow a minimum of 3 to 7 days for the concrete to cure (at the discretion of the owner/agency) prior to installing bearing devices and lowering the superstructure.
9. Install bearing devices.
10. Lower the superstructure back into proper position.

Particular attention should be given to older unreinforced bearing seats. When unreinforced bearing seats start to deteriorate, rapid deterioration from freeze and thaw cycles can produce substantial bearing loss over a short period. This is because the concrete in unreinforced bearing seats is not confined by the rebar.

13.4.10 FRP Repairs

Fiber reinforced polymers (FRP) fabrics are used to increase the axial and shear capacity of concrete columns and also to protect columns against intrusion of corrosive and deleterious materials. In seismic areas, these fabrics are primarily used to strengthen columns against earthquake loads. For column wrapping, FRP with carbon fibers are usually preferred to FRP

with glass fibers. Although slightly more expensive, FRP with carbon fibers offer higher tensile strength, i.e., 350 ksi versus 220 ksi.

It is important that prior to wrapping a column, the condition of the concrete is assessed for any evidence that sound concrete may be critically contaminated with chlorides. If chlorides are present in the sound concrete, there is potential for corrosion of the reinforcing steel and deterioration of column after it is wrapped, which may be difficult to inspect. Thus, if tests show that concrete is critically contaminated with chlorides, one of the following measures may be taken:

- Do not wrap the column.
- Extract the chlorides from the column by electro-chemical methods prior to wrapping the column. This procedure requires a specialized contractor and can take up to 2 months.
- Admix a corrosion inhibitor in the repair/patch concrete. Subsequent to repair, apply an effective spray-on type corrosion inhibitor on all unrepaired surfaces. Let the surface completely dry before wrapping the column. This procedure is only recommended when the concrete is not heavily contaminated with chlorides. An engineer should assess the condition of the concrete and approve wrapping the column. The corrosion inhibitor manufacturer recommendations should be closely followed.

A clean concrete substrate is essential to the effectiveness of the FRP system in achieving the design strength and the intended design objectives. It is necessary to completely clean the substrate of any bond inhibiting materials and residue to accommodate the successful application of the FRP system

Cleaning may be performed with blast cleaning, air blower, pressure washing or other equivalent means. Air cleaning equipment must be equipped with oil traps. The cleaned surface must be protected against re-deposit of any bond inhibiting materials prior to the application of the FRP system.

All defective areas of concrete must be removed prior to installing a FRP. First examine the existing conditions to assess the quality of the concrete substrate, identify potential obstructions, and verify the dimensions and geometries necessary for placement. Defects may include loose and broken debris or delaminated and spalled sections of concrete, voids and honeycombs, and deteriorated concrete. Any void larger than 1/2 inch in diameter and depth should be filled with repair material. The repair material should have strength greater to, or equal to the strength of the original concrete, but no less than 4.5 and 5.5 ksi at 7 and 28 days. If moisture intrusion is significant, water protection, weep holes, and water conveyance must be provided before full reconstruction.

FRP wraps are only effective when they are fully bonded to the substrate. FRP is considered a bond-critical application. If the bond fails, so does the FRP repair. Checking for bond can be as simple as looking for air bubbles and pockets to using nondestructive testing such as ultrasonic scanning or infrared thermography. Most construction specifications also require sample test specimens that can be sent to the laboratory where destructive bonding tests can be performed.

To ensure the FRP is in full contact with the substrate, all surfaces should be freshly exposed and free of loose or unsound materials (abrasive or water-blasting techniques). Any localized

out-of-plane variations (form lines) should not exceed 1/32 inches. All irregular surfaces or variations should be ground away or smoothed over with an epoxy putty.

All cracks in the substrate wider than 0.01 inch should be filled using pressure injection of epoxy according to specifications (see section on crack repair). The crack should be free of loose, unsound or bond inhibiting materials such as oil, efflorescence or moisture. NCHRP Report 514, ACI 546R, and ICRI 03730 list recommendations and provides guidance for the installation of bond critical applications.

Movement of cracks wider than 0.01 inch may cause delamination or fiber crushing in externally bonded FRP systems. Crack injection helps restore concrete strength and prevent water leakage behind the FRP system.

Ambient and concrete surface temperatures should be within 50 ^{o}F to 95 ^{o}F or as per the resin manufacturer recommendations. Moisture levels on all contact surfaces should be less that 10 percent at the time of installation of the FRP system. Moisture on fiber sheets can cause problems with wet-out and cure of the system. Surface moisture should be measured with a mortar moisture meter, or alternatively, an absorbent paper.

Application of the system should not proceed if any moisture vapor transmission is present. Moisture vapor transmission from concrete surface through uncured resin may cause air pockets and surface bubbles, compromising the bond between the FRP system and the concrete. Any bubbles that develop from moisture vapor transmission can effectively be injected with an epoxy filler. However, some manufacturers use moisture to activate the bonding resin. All applications should follow the instructions of the manufacturer.

In most applications, the FRP system can be applied while the structure is in service. Shoring may be provided to either support the existing structure prior to repair, or to reduce its initial deflections prior to strengthening. Shoring can also be used to induce an initial camber in the system, effectively stressing the FRP system.

Necessary equipment for installation will vary depending on the system requirements. The equipment may include:

- Resin Impregnators
- Rollers
- Sprayers
- Lifting/Positioning Devices

Suggested Procedure

Column Wrapping

Note: Manufacturer guidelines shall be followed for any specific application.

1. Surface preparation and column wrap should include the full exposed column height plus 2 feet below ground.
2. All concrete surfaces should be repaired (including spalls and delaminations) and epoxy injection crack sealing performed. Surface should be free of sharp edges that can damage the fabric. Surface voids and depressions should be filled with epoxy.
3. Surface should be completely dry at the time of application of fabric. Newly repaired or patched surface should be cured at least 7 days prior to wrapping.
4. One prime coat of manufacturer's epoxy should be applied to the surface and should be allowed to become tacky to touch.
5. Fabric, usually about 2 to 3 feet wide, should be saturated at the job site with resin (usually epoxy) as per the specified fiber-resin ratio.
6. Saturated fabric should be wrapped around the column by hand lay-up, using methods that produce a uniform, constant tensile force that is distributed across the entire width of fabric.
7. Entrapped air, if any, should be rolled out before the resin sets.
8. Subsequent layer(s) should be applied continuously (or spliced) until specified number of layers is achieved at a section. Adjacent sections should utilize a butt joint.
9. The system should be protected against water and rainfall for at least 4 days following installation.
10. After the system is cured, a protective/aesthetic topcoat is applied to the fiberwrap surface.

Wet lay-up systems may be installed using dry or pre-pregnated fiber sheets and saturants. Alternatively, wet lay-up systems can be applied using special equipment, such as a saturator, which will automate and speed up the installation process.

The term resin is a generic nomenclature used to identify all polymers employed in wet lay-up systems. Depending on its function, resin is more specifically identified as primer, putty and saturant. It should be noted that not all FRP systems utilize putty. Care should be taken when fully mixing the two-part resins, as excessive agitation using electric mixers may cause froth and bubbles to form, resulting in voids in the mixture. The viscosity of a mixed resin that has exceeded its pot life will continue to increase, adversely affecting its ability to penetrate the concrete surface or saturate the fiber sheet.

Primer is to be applied to the concrete surface to penetrate the open pores using a brush or a clean roller. The primer will effectively help hatch and strengthen the external layer of concrete and will improve the bond between the concrete substrate and the FRP system. The primary function of the putty (if used) is to provide a smooth concrete surface. Adding silica sand to the

putty may improve its stability and prevent swelling. The resin that impregnates the fibers is the key component to form the FRP laminate that repairs or retrofits the concrete member. The rate of coverage of the resin is listed on the materials data sheet supplied by the manufacturer. The amount generally depends on the type of resin, the ambient temperature, and the fiber sheets prior to curing. This installation procedure is applicable to a single fiber sheet or the first fiber sheet or ply in a multi-ply application, as shown in Figure 13.47 Alternatively, the fiber sheet may be separately impregnated using a resin-impregnating machine before placement on the concrete surface.

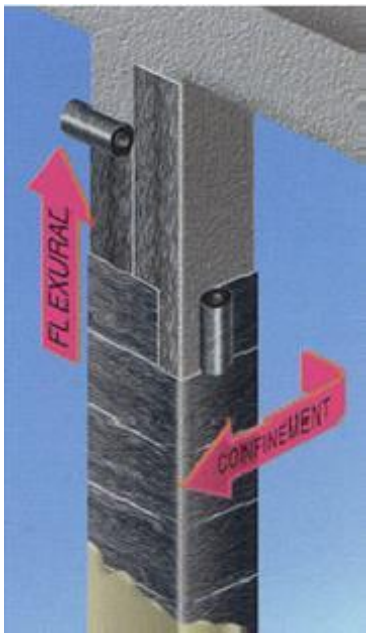


Figure 13.47 Advantages of Multi-Ply FRP Application

For ease of handling and to avoid wrinkling, fiber sheets are typically cut in segments shorter than 15 to 20 foot lengths. Metal serrated rollers are often used to force resin between the fibers and to remove entrapped air. However, caution should be taken not to use excessive forces, as rupture of the fibers could occur. Equally important is that rolling must be performed parallel to the fiber direction.

Given the vast number of such systems, the method of placement can vary by manufacturer. It is best to follow the recommendations of the manufacturer regarding the timing and sequence of stacking, overlap and banding, horizontal and vertical joints, staggering of splices and overlap and butt joints.

Use appropriate safety protection while cutting pre-cured sheets. Use appropriate safety protection while cutting pre-cured sheets.

The FRP system should be allowed to cure as recommended by the manufacturer. Curing is a time and temperature dependent process and can take up to several days in ambient temperature. In some FRP systems, pressure must be continuously applied through external means to prevent sag and pull-off during cure. Temporary protection in the form of plastic screening or tenting may protect the system while installation or curing is underway. Rain, dust, dirt, excessive sunlight, extreme temperatures, high humidity and vandalism are common threats that can be lessened with protection during construction.

Sharp corners should be rounded by grinding to a minimum 3/4 inch radius (see Figure 13.48). Rounding the corners prevents stress concentrations and voids between the FRP and the concrete. Surface irregularities should be ground smooth and all surfaces should be flat or convex to ensure full bonding of the FRP to the substrate. Large voids should be filled with a repair material compatible with the existing concrete.

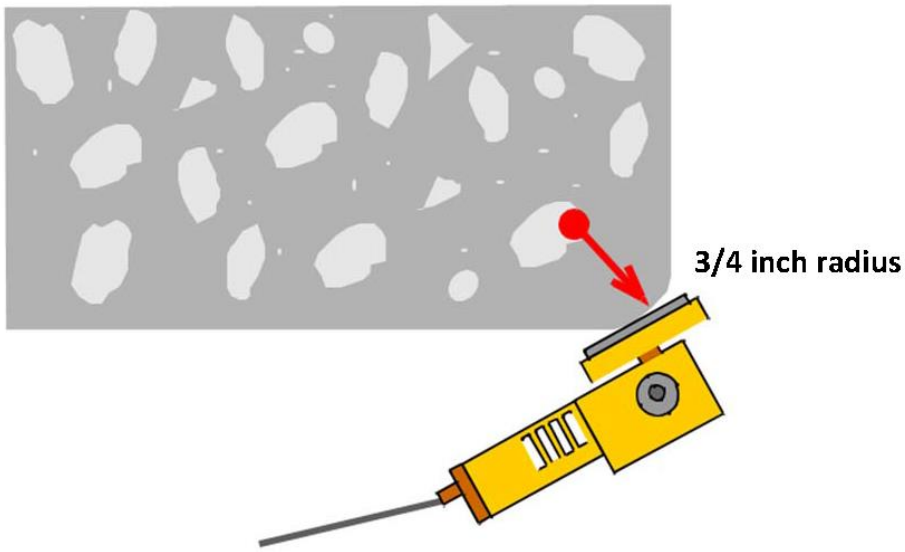


Figure 13.48 Grind Sharp Corners to 3/4 Inch Radius (Courtesy of MDOT)

Various applications for the use of FRP in the repair and rehabilitation of concrete substructures are shown in Figure 13.49 through Figure 13.51.

Figure 13.49 Strengthening a Spandrel Arch with FRP

Figure 13.50 Pedestal Repair with FRP

Figure 13.51 Completed Column Repair with FRP

13.4.11 Post-Tensioning Piers and Bents

Concrete pier caps may develop cracking due to flexure and/or shear. External post-tensioning may be applied to reduce crack growth rate and to strengthen the cap. A competent engineer should design this type of repair before the repair is initiated.

When to Call the Engineer

Call the Engineer when planning external post-tensioning repairs.

Suggested Procedure

Application of Post-Tensioning Piers

1. Epoxy inject all cracks in the pier cap and allow the epoxy to harden.
2. Obtain steel cables (or rods) for post-tensioning and PVC, HDPE, or polypropylene pipes as ducts for the bars for corrosion protection. Note that HDPE or polypropylene piping is preferred over PVC for greater environmental resistance and higher ductility.
3. Erect all PVC pipe supports at both sides of the pier cap. Locate existing stirrups and horizontal bars in the cap prior to installing expansion bolts for the supports.
4. Erect PVC pipes (with post-tensioning steel inside) to the supports on the pier cap.
5. Erect post-tensioning anchorage assembly at both ends of the pier cap.
6. Post-tension the system to the required load per bar. Use a sequence to balance the loads during post-tensioning.
7. Grout the ducts and seal the post-tensioning anchorage to prevent moisture penetration which can cause corrosion.

An example of external post-tensioning rods used in pier containment is shown in Figure 13.52.

Figure 13.52 External Post-Tensioning Rods used in Pier Containment

13.5 Repair and Rehabilitation of Steel Substructures

Steel substructures can consist of piling, pile bents, frames and towers. Examples are included in Chapter 3. Most steel substructures utilize bolting or welding in the original design and these connection types must be considered in repair and rehabilitation.

13.5.1 Welding

It is common practice to use welding for shop fabrication of steel members and for welding pieces in preparation for maintenance activities. Field welding is relatively difficult to perform

properly and requires individuals with the necessary skill and physical ability. It is equally difficult to properly inspect welds that carry major stresses under field conditions. In the shop, automatic machines make the majority of the welds and the rest are made under ideal conditions. Highly sophisticated equipment using X-rays, gamma rays, or ultrasonics can also be used to inspect the welds.

Welding is the process of joining pieces of steel plate together to form one cohesive piece. For this to occur the steel areas to be joined are heated to the point of becoming almost fluid, at which time weld material is added as a sort of glue to join the pieces together. Welding is so sophisticated since the heat affected zone around the weld can actually cause cracking if it is heated too much or allowed to cool too fast. Insulating material is usually automatically applied to the weld to help it cool down properly but that material can be affected by outside conditions such as wind. The process, if performed correctly, will make the steel act as one bridge element. If not, the weld can cause premature cracking and dramatically reduce the service life of the bridge.

Often during rehabilitation, widening and strengthening projects, it is necessary to make field welds. This operation requires proven expertise. The welder must be tested and have a certificate of qualification. It is also important that the qualification cover the type and position of field weld to be performed. A structural engineer should review the operation to ensure the strength or structural integrity of the joining members are not altered by the connection or the heat of the weld.

Deteriorated steel can be repaired by adding sections when there is sufficient working space to weld or bolt them in place. Loss of section should be evaluated by an engineer to determine if repair or replacement is warranted. The web and flanges of the steel section can be strengthened by welding steel plates extending far enough above and below the deteriorated area to carry the full load on the pile.

When to Call the Engineer

Call the Engineer when evaluating section loss and any welding.

Welded repairs should not be performed unless sufficient knowledge of the existing steel is available to know if the steel is weldable. Chemical tests can be used to determine whether unknown steel is weldable. The chemical composition of a metal is important in determining its weld-ability. The likelihood of cold cracking depends on the carbon equivalent (CE) of the steel. When the CE is below 0.55 percent, cold cracking is not likely and no special precautions are needed for field welding. When the CE is above 0.55 percent field welding should only be performed by a welder experienced in field welding of bridge repairs. Any field welding should be performed a certified welder and should not be performed on members where fatigue of the weld could be a problem. Poor quality welded repairs on tensile members can actually do more harm than good.

Field welding is more likely to result in flaws than shop welding due to a lack of control of such factors as moisture, temperature and welder access. Bolting is always preferred over field welding.

There are situations where welding is used by bridge crews for the following types of repairs:

- Repairing defects such as nicks and gouges
- Adding reinforcement segments such as plates
- Adding bracing members
- Replacing a portion of a steel member

For field welding of piles, where extreme section loss is present in the pile at the interface with the footing, repairs can be made by welding plates to form an angle with one leg against the footing and the other against the pile. Stiffeners should be placed across the angle as necessary. These angle plates should be placed on both flanges. When the welding is completed, all of the exposed piles should be given a heavy, protective coating. Fill should then be placed around the piles up to the bottom of the footing.

13.5.2 Bolting

There are thousands of bridges still in service which have riveted connections. The majority of these structures have been in service for 50 years or more, since riveting was a relatively common fabrication procedure until about 1960 or so. During the period when riveting was being phased out in favor of welding, a number of bridges were fabricated using shop rivets with high strength bolts used for field connections. Corrosion can build up between the plates of the riveted members. The corrosion then expands and pops off the rivet heads. Where this occurs, repairs are normally made to the connections with bolts. If a rivet has popped off, the other rivets in the area of the failed rivet should be checked by tapping with a hammer. Good rivets will ring, bad rivets will thunk. This may save having to go back to the same location after the next inspection to replace additional rivets.

High strength bolts should be used to replace deteriorated rivets. If rivets are found to be deteriorated to the extent that they do not contribute to the strength of the connection, the connection should be analyzed by a competent engineer to determine if the member should be jacked to restore the original dead load distribution. If only a handful of rivets require replacement, they may be replaced individually with no special procedure. If only one rivet has popped off, the other rivets in the area of the failed rivet should be checked by tapping with a hammer. Good rivets will ring, bad rivets will thunk. This may save having to go back to the same location after the next inspection to replace additional rivets.

When a connection must be completely re-fastened, begin at the center of mass of the connection, and proceed outward in a balanced manner. Bolting may be used as a repair method or as a supplement to other methods. Replacement of damaged elements with a new piece of steel fastened with high strength bolts is a recommended repair method. Procedures are discussed in this section on the proper pre-tensioning of the bolt to ensure a proper clamping force of the steel plates to prevent movement. However, the ultimate strength of a connection is independent of the bolt pretension and slip movement.

Figure 13.53 shows the markings required on the bolts as per ASTM Specifications. Certain markings are mandatory. In addition to the mandatory markings, the manufacturer may apply additional distinguishing markings.

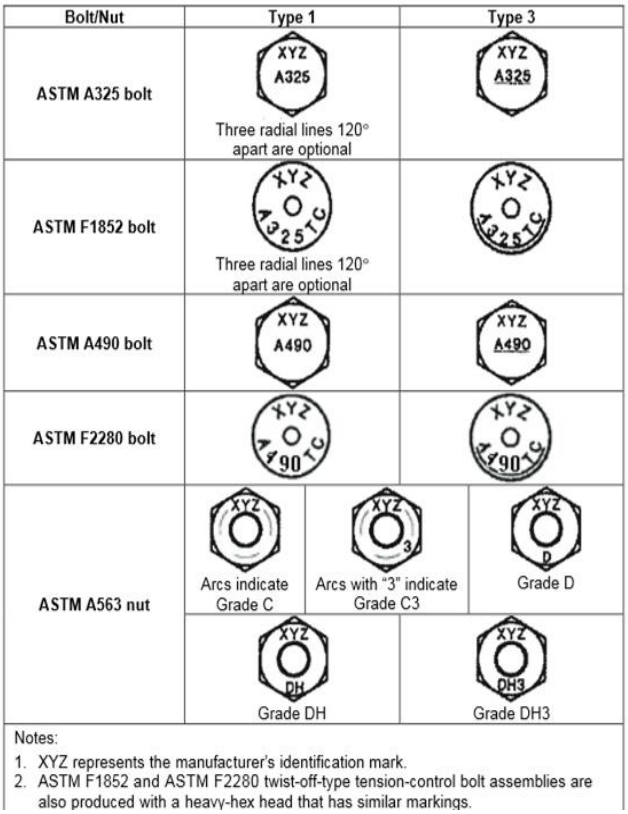

Bolt/Nut
Type 1
Type 3
ASTM A325 bolt
XYZ A325
Three radial lines 120° apart are optional
ASTM F1852 bolt
XYZ A325TC
Three radial lines 120° apart are optional
ASTM A490 bolt
XYZ A490
ASTM F2280 bolt
XYZ A490TC
ASTM A563 nut
Arcs indicate Grade C
Arcs with "3" indicate Grade C3
Grade D
Grade DH
Grade DH3
Notes:
1. XYZ represents the manufacturer's identification mark.
2. ASTM F1852 and ASTM F2280 twist-off-type tension-control bolt assemblies are also produced with a heavy-hex head that has similar markings.

Figure 13.53 ASTM Heavy-Hex Nuts Marking Standards (Courtesy of The Research Council on Structural Connections)

The principal features of heavy-hex structural bolts that distinguish them from bolts for general application are the size of the head and the unthreaded body length. The head of the heavy-hex structural bolt is specified to be the same size as a heavy-hex nut of the same nominal diameter so that the ironworker may use the same wrench or socket either on the bolt head and/or on the nut.

The nominal dimensions of standard holes for high-strength bolts is 1/16 inch larger than the bolt installed.

Washers provide a hardened non-galling surface under the turned element, particularly for torque-based pretensioning methods such as the calibrated wrench pretensioning method and twist-off-type tension-control bolt pretensioning method. Circular flat washers that meet the requirements of ASTM F436 provide both a hardened non-galling surface and an increase in bearing area that is approximately 50 percent larger than that provided by a heavy-hex bolt head or nut.

All bolts, whether black, galvanized, or weathering steel, must be lubricated when installed. Most manufacturers apply a water-soluble oily lubricant to black bolts, nuts and washers as a part of their production operations. If the fasteners are exposed to rain, snow, dew, condensation or other moisture conditions, this lubricant may be washed off. This lubrication will also evaporate after a period of time when left in open containers. Bolts and nuts should not be stored in exposed weather. The most effective lubrication is placed on the threads of the bolt, the threads of the nut and the inside face of the nut.

If a bolt, nut or washer has lost its lubrication, it is important that the component be re-lubricated prior to installation. The type of lubrication to be used is not specified, but typically a similar oil-based product, stick wax, bee's wax, liquid wax or spray lubricant can be used.

Should any of the bolts, nut or washers show rust, the rust must be cleaned from the surface of the fastener component and then be re-lubricated. Dirt, sand, grit and other foreign material must be cleaned off the bolts prior to installation, with re-lubrication when necessary.

Bolts are first brought to a “snug tight” condition. “Snug tight” is defined as “the tightness achieved with the full effort of a worker with an ordinary spud wrench or a few hits of an impact wrench that brings the connected plies into firm contact”. Once “snug tight”, a second application of force brings the tension on the bolt to the required loading. For field applications, ensuring that the final application of force brings the force in the bolt to the required loading can be determined by several methods: “turn of the nut”, torque or calibrated wrench, twist-off bolts, and Direct Tension Indicators (DTIs) are commonly available.

The most common technique for pre-tensioning (or pre-loading) a bolt is the “turn-of-the-nut” method. This method applies an additional rotation of the wrench a predetermined amount after the nut has been tightened to a snug fit. This method involves tightening the fastener to an initial "snug tight" condition and then applying a prescribed amount of turn to develop the required preload. The actual preload will depend on how far the nut is turned as well as how much preload was established prior to the turning.

Suggested Procedure

Turn of the Nut

1. Snug the joint to bring the assembly into firm contact. Apply a few impacts with impact wrench until solid sound or apply full effort with a spud wrench.
2. Inspect the joint to verify "snug tight".
3. Match mark bearing face of the nut and end of the bolt with a single straight line.
4. Using a systematic approach which would involve the appropriate bolting pattern, apply the required turns as shown in Table 13.2. The amount of turn past snug tight is based on the ratio between the bolt length and diameter, as well as the slope disposition of the outer steel plies.

Table 13.2 Turn of the Nut Rotation Table Using Condition Under Bolt Head and Under Nut

Bolt Length	Both faces flat (normal to bolt axis)	One face sloped, but not more than 1:20	Both faces sloped, but not more than 1:20
Less than or equal to 4D	1/3 Turn	1/2 Turn	2/3 Turn
More than 4D and less than or equal to 8D	1/2 Turn	2/3 Turn	5/6 Turn
More than 8D and less than or equal to 12D	2/3 Turn	5/6 Turn	1 Turn

Once the bolt is “snug tight”, a mark on the wrench socket is aligned with the marker line “x”. The sequence is shown in Figure 13.54. A mark is made on the nut and extends onto the end of the bolt. This allows the inspector to see that the nut has been turned relative to the bolt the prescribed amount, in this case ½ a turn, to the “y” mark.

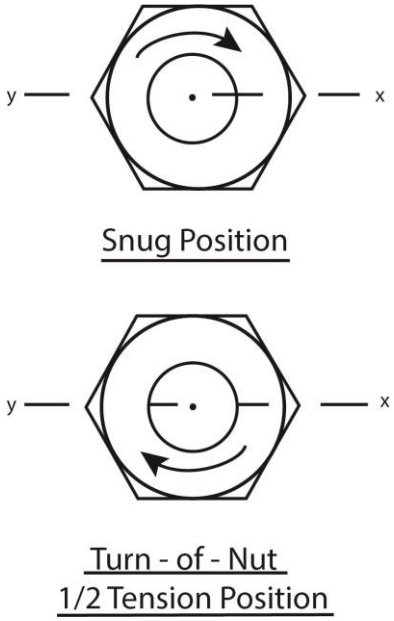


Figure 13.54 Turn of the Nut Method

Torque or calibrated wrenches measure the torque required to turn the bolt; they do not measure the tension on the bolt. It is generally a specification requirement that fasteners be oily to the touch prior to being installed. When compared to oily fasteners, bolts that have lost their lubrication may require as much as twice the torque to install them, and their ability to reach their proper tension force is questionable. An unlubricated bolt and nut system when tightened may strip, twist and even fracture.

In some cases for bolts, re-lubrication mandates the retesting of fasteners in a Skidmore-Wilhelm (or similar device) prior to installation in the structure. This verifies the effectiveness of

the re-lubrication. Highly efficient lubricants can actually increase the risk of thread stripping, so this condition is also checked.

A correctly calibrated wrench is critical for a proper connection. Re-calibration must be done daily before work begins or:

- When the lot of any component of the fastener assembly is changed
- When the lot of any component of the fastener assembly is re-lubricated
- When significant differences are noted in the surface condition of the bolt threads, nuts or washers
- When any major component of the wrench including lubrication, hose and air supply are altered

Twist-off-type tension-control bolt assemblies have a splined end that extends beyond the threaded portion of the bolt. During installation, this splined end is gripped by a specially designed wrench chuck and provides a means for turning the nut relative to the bolt. Twist-off-type tension-control bolt assemblies must be used in the as-delivered, clean, lubricated condition.

Many twist-off bolts use a special lubricant that is not as oily as common structural bolts. Contacting the manufacturer of the twist-off bolt system is encouraged prior to re-lubrication. These fasteners are particularly sensitive to inadequate lubrication and over-lubrication, and loose bolts or broken bolts may result.

To measure bolt tension another option is use of direct tension indicators (DTIs). A DTI is a washer like device that is placed on one or the other end of all bolts, and after snugging the joint by partially (but not fully) compressing the DTI, all the DTI's are "crushed" to the point where a feeler gage cannot be inserted half way around, or to the point where the silicone pockets have been squeezed out for “squirter” style DTIs. DTI's are completely independent of the torque resistance of the bolt assembly, and in theory by simply visually inspecting the DTI ‘bumps’ you should be able to verify bolt tension. However, there can be installation issues and it is recommended that a feeler gauge be used to verify that the DTI has been fully compressed.

During installation, care must be taken to ensure that the direct-tension-indicator arches are oriented to bear against the hardened bearing surface of the bolt head or nut, or against a hardened flat washer if used under turned element, whether that turned element is the nut or the bolt. This is so that the arches are deformed by compression only and not by galling. The torque tension relationship is based solely on the compressibility of the arches. Inspection by using a feeler gage (on a sample of the bolts only) can be done by anyone at any time. If the DTI is put on the nut end of the bolt, tightening can be done by one person because it is not necessary to hold the bolt roll. Final tensioning should be done as per a pre-described bolt tightening pattern.

13.5.3 Splicing

Splicing involves adding additional steel attached by bolts to a defective area of a member to transfer a load around that defective member. Steel H-piles may be damaged particularly if located in waterways where they may be struck by heavy barges or near roadways as in work zones where they may be struck by heavy equipment. Damage in the form of bent, torn or cut flanges may effectively reduce the cross section and load-bearing capacity so that repair must

be performed. A steel pile that cannot be easily supplemented because of access or scheduling may be strengthened by repairing with bolted channels as a temporary measure.

When to Call the Engineer

If splicing new steel sections to damaged steel, contact the Engineer to determine the appropriate size, number, and spacing of the bolts.

Planning should include the following:

- Select appropriate channel size to meet strength and dimensional requirements.
- Determine length of damaged area and secure steel channels of selected size that have been fabricated in appropriate lengths with necessary hardware.

Equipment and tools necessary will include:

- Equipment for drilling bolt holes
- Protective coating material
- Necessary staging

Suggested Procedure

Steel Pile Strengthening by Splicing

1. Clean damaged pile.
2. Locate extreme limits of deteriorated section.
3. Channel section should be 18 inches longer than the distance between these limits.
4. Thoroughly clean area to which channel is to be bolted.
5. Clamp channel section in place against pile.
6. Locate and drill holes through channel and pile for high strength bolts.
7. Place bolts and secure.
8. Remove clamps.
9. Coat with protective coating.

An example of steel pile strengthening is presented in Figure 13.55.

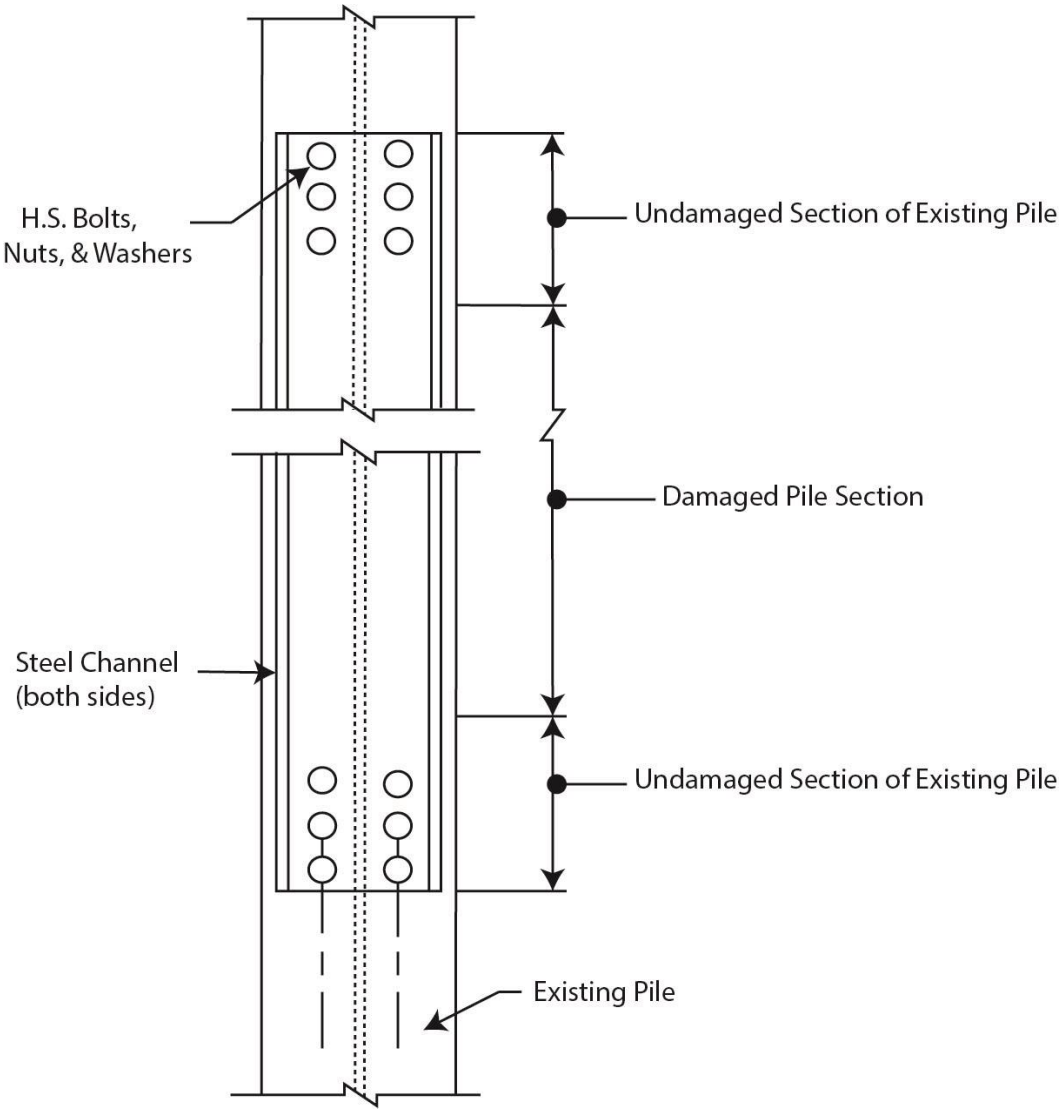


Figure 13.55 Steel Pile Strengthening

13.5.4 Jacketing

Pile jacketing of steel piles is basically the same as that described previously for concrete piles. Both flexible and rigid forms can be used. Often fiberglass and plastic forms are used because of ease of erection for underwater applications. Prior to pile jacketing, marine growth and corrosion are cleaned.

Stands-offs are placed on the pile flanges before the forms are installed and concrete is placed. Figure 13.56 shows stand-offs inside a flexible form. Welded wire fabric is typically used to reinforce the concrete against cracking. Concrete jackets can cause accelerated corrosion on a steel pile when both concrete and water are in contact with the steel. A corrosion cell will develop either below the bottom of jacket or above the top of jacket. Concrete jackets should be extended well into the mudline and, well above the high waterline. Cathodic protection could be included in the repair.

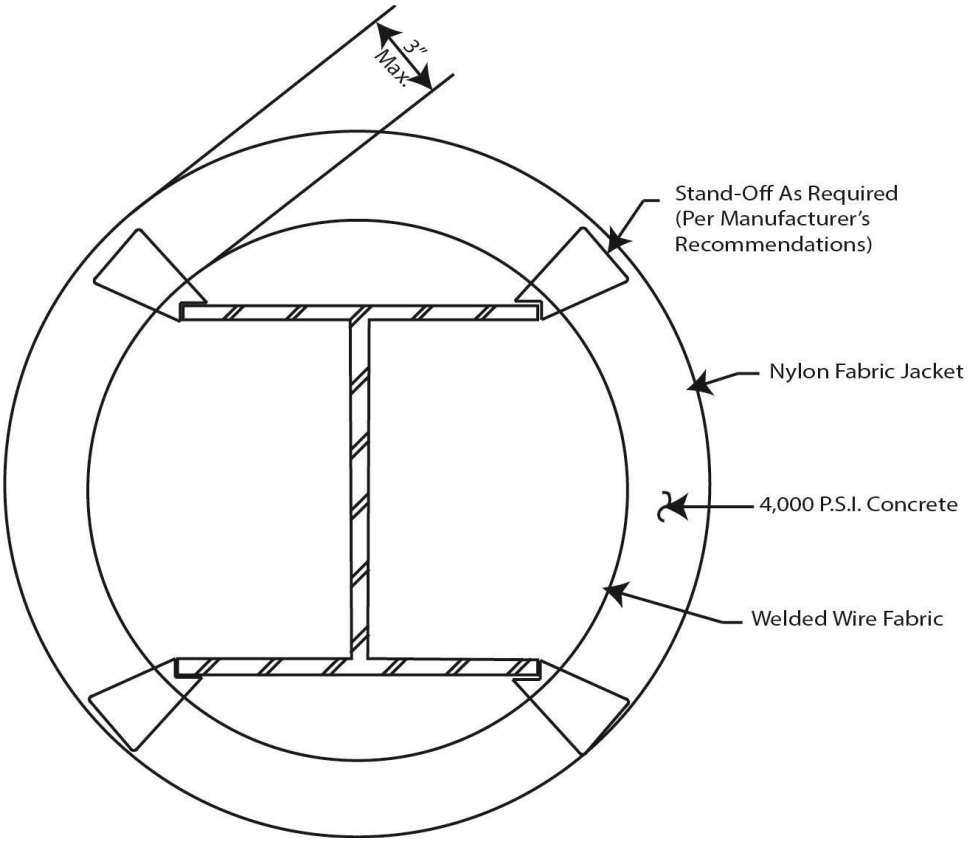


Figure 13.56 Flexible Formed Pile Jacket

When steel piles require additional support or protection, an integral pile jacket can be placed around the steel piling. The encasement of the steel piles is accomplished by filling a fiberglass form with Portland-cement grout. After the concrete hardens, the fiberglass form remains in place as part of the jacket. The integral jacket provides protection to steel piles above and below the water. If the pile has deteriorated to the point that additional steel support is required, cover plates can be added to the pile prior to placing the jacket or a reinforced concrete jacket can be designed.

Suggested Procedure

Installation of Integral Pile Jacket

1. Clean the steel surfaces by sandblasting the surfaces clean of oil, grease, dirt, and corrosion (near white metal).
2. Place the pile jacket form around the pile.
3. Ensure standoffs are attached to the form.
4. Seal all joints with an epoxy bonding compound and seal the bottom of the form to the pile.
5. Brace and band the exterior of the form to hold the form in place.
6. Dewater the form. Fill the bottom 6 inches of the form with epoxy grout filler.
7. Fill the form to within 6 inches of the top with a Portland cement grout filler.
8. Cap the form with a 6-inch fill of epoxy grout.
9. Slope the cap to allow water to run off.
10. Remove the external bracing and banding and clean off the form of any deposited material.

All exposed steel should be protected against corrosion. Steel piles must be protected by coatings that prevent the dissolved oxygen in the water from contacting the steel. Epoxy coating systems and polyvinyl chloride barriers have been used. The portion of a steel pile in the water and beyond the concrete jacket has especially potential for galvanic corrosion. Zinc anodes can be attached to H piles to abate corrosion of steel in salt water as shown in Figure 13.57.

Small anodes are used when less than 8 feet of pile is exposed. Large anodes are used when greater than 8 feet of the pile is exposed. It should be noted that anodes will be consumed in time and the life of the anodes will depend on its weight.

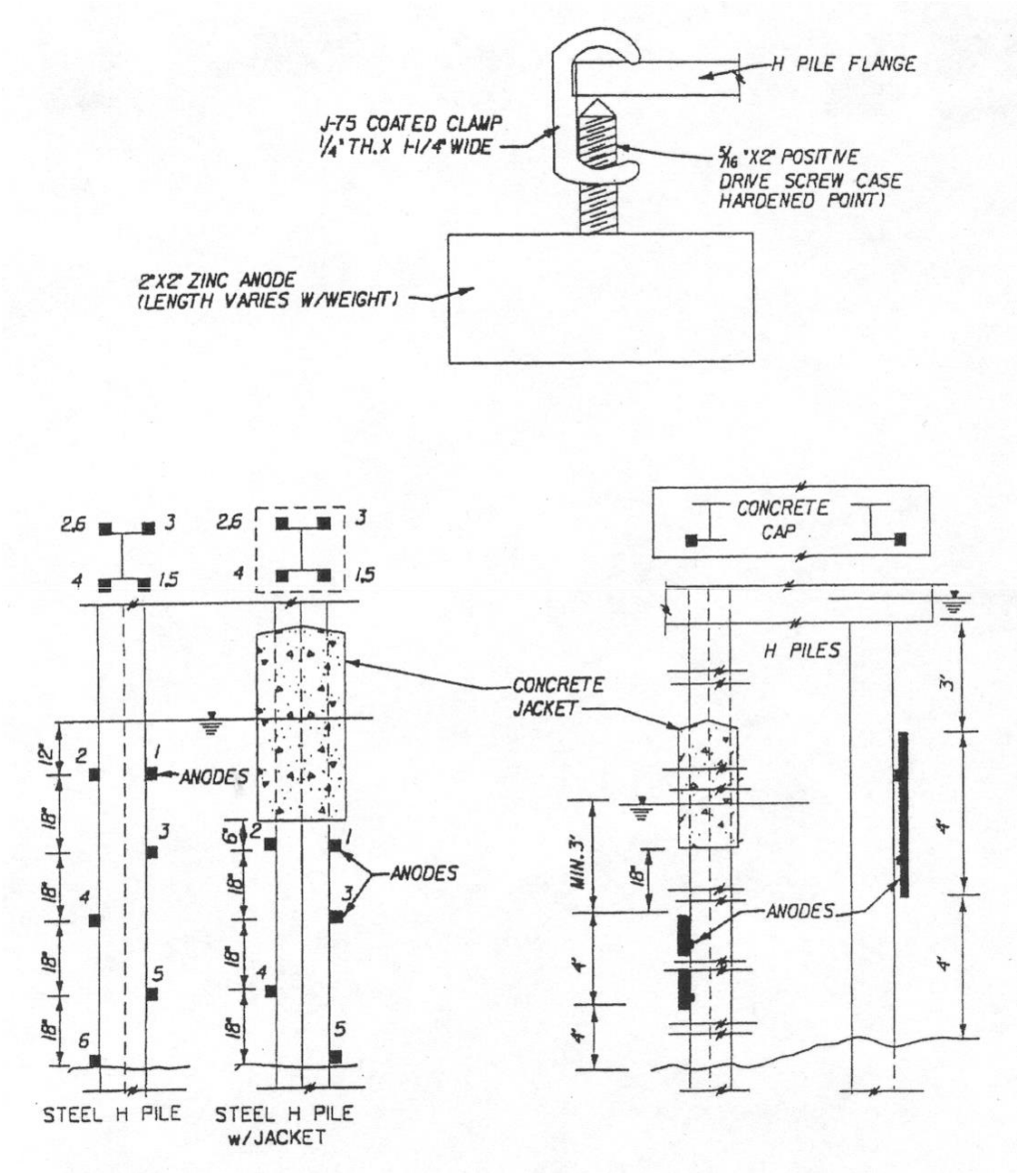


Figure 13.57 Corrosion Protection of H Pile by Zinc Anodes

13.5.5 Supplemental Bracing

As truck loads continue to get bigger, the demand on many aging bridges increases. Slender steel structural elements widely used in bridge superstructures and braced substructures to resist these loads are built primarily for tension, however, and can buckle under compressive loads.

Supplemental bracing can be used to increase the allowable capacity of a member. The bracing reduces the unsupported length and adds more material to the existing member.

13.5.6 FRP Repairs

When steel piles require additional support or protection, an integral pile jacket can be placed around the steel piling. The jacket can be made by filling an FRP form with Portland-cement grout or FRP wrapping as shown in Figure 13.58.

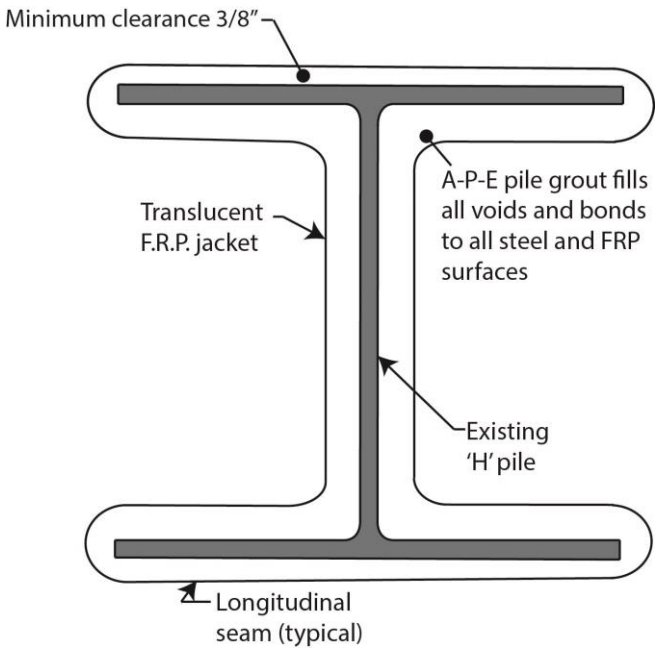

Minimum clearance 3/8"
Translucent F.R.P. jacket
A-P-E pile grout fills all voids and bonds to all steel and FRP surfaces
Existing 'H' pile
Longitudinal seam (typical)

Figure 13.58 FRP Jacket for Strengthening Steel Piles

13.6 Repair and Rehabilitation of Timber Substructures

A review of the causes of deterioration in timber members should be conducted before starting the repair operation. Otherwise, new timber members will be exposed to the same conditions that caused the original problems. The conditions should be evaluated and steps taken to prevent the same damage from occurring to the new member. The new member should be treated with preservative when an extended service life is required. Adjacent and contacting members should also be treated with preservative when an extended service life is required. And all adjacent and contacting members should be checked for possible decay infections. Such areas, if confirmed or questionable, should be replaced, treated in place, or the new member protected from them.

The repair or replacement of timber caps on either abutments or intermediate timber bents is a common repair procedure in the maintenance of timber substructures.

The conditions which result in a cap failure can be decay from a buildup of dirt on the cap. But, in a surprising number of instances, the cap fails as the abutment transfers earth pressure into the superstructure. This can be first observed as a twisting of the end of the cap. Many times this is mis-diagnosed as merely warping and twisting of the timber cap. Unfortunately, what is happening is that the cap is being restrained at the top by the superstructure and is being subjected to a large lateral load at the bottom of the cap by the piling. The pilings are being pushed by the earth pressure behind the abutment. As the cap starts to twist from these opposite forces it will start to crack. This will let moisture into the center of the member and decay will follow.

When inadequate residual strength remains in a member, replacement or reinforcement is necessary. This can be accomplished by replacing the entire member, replacing a portion of the member, or adding a sister member or reinforcing element to carry the load. Obviously, these

operations require a thorough structural evaluation to ensure the bridge is properly supported during the repair or rehabilitation procedures and that the new elements are adequate.

It may be more practical to remove only the defective part of a member. In such cases, the recommendations above for replacing the member also apply. Additionally, it is important to remove an adequate length of the defective member to ensure elimination of the source of the problem. The undetectable extensions of the infection fungi may reach 6 to 12 inches in the grain direction beyond the apparent limits of decay. A safe rule in removing decayed parts of members is to include visible decay plus an additional 2 feet of the adjacent wood in the grain direction. All newly exposed, cut-off timber faces should be treated with a wood preservative.

The decayed top of a wood bearing pile can be repaired by cutting off the damaged portion and fumigating the exposed cut-off to destroy any remaining affected material. A section of sound timber is installed and secured with epoxy cement or drift pins and fish plates to build the pile to the proper height. An example of pile top replacement is shown in Figure 13.59.

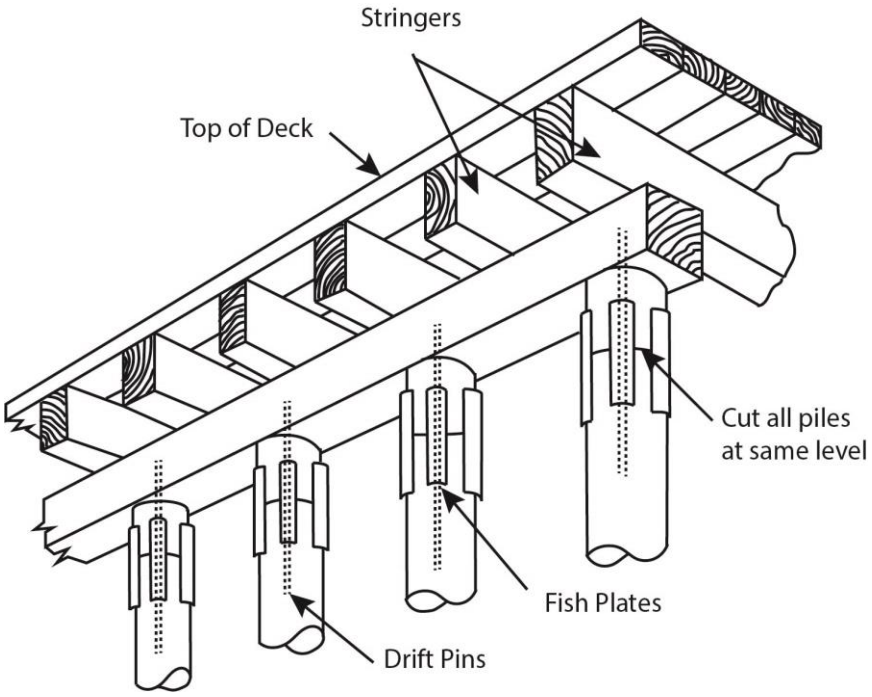


Figure 13.59 Replacing Tops of Piles

13.6.1 Jacketing

Jacketing is a common preventive maintenance technique for timber piles that can also be used in the repair of moderately damaged piles. In general, the equipment and techniques for jacketing timber piles is similar to jacketing concrete or metal piles.

A jacket is placed around the pile as shown in Figure 13.60. Reinforcing steel is placed and the jacket is filled with concrete. The reinforced concrete acts to increase the pile strength if designed properly.

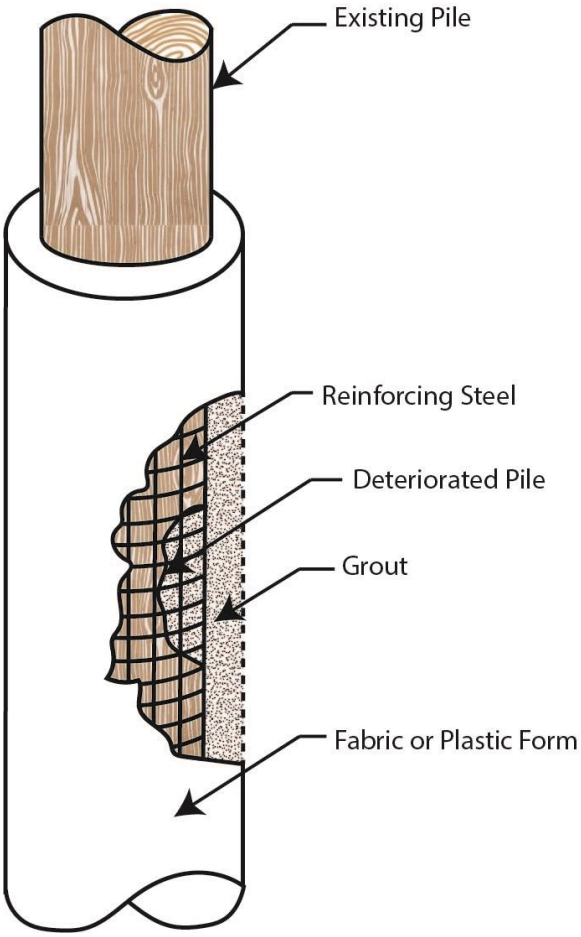


Figure 13.60 Timber Pile Jacketing

13.6.2 Scabbing and Splicing

Timber repairs usually involve the addition of steel or wood to strengthen deficient members. Scabbing refers to techniques to add effective section to reinforce deteriorated or deficient timber bridge members. The material, steel or timber, is generally attached by bolts to the defective area to transfer a load around that defective member over a substantial length. Splicing refers to similar techniques for repair over a shorter length. Details are shown in Figure 13.61.

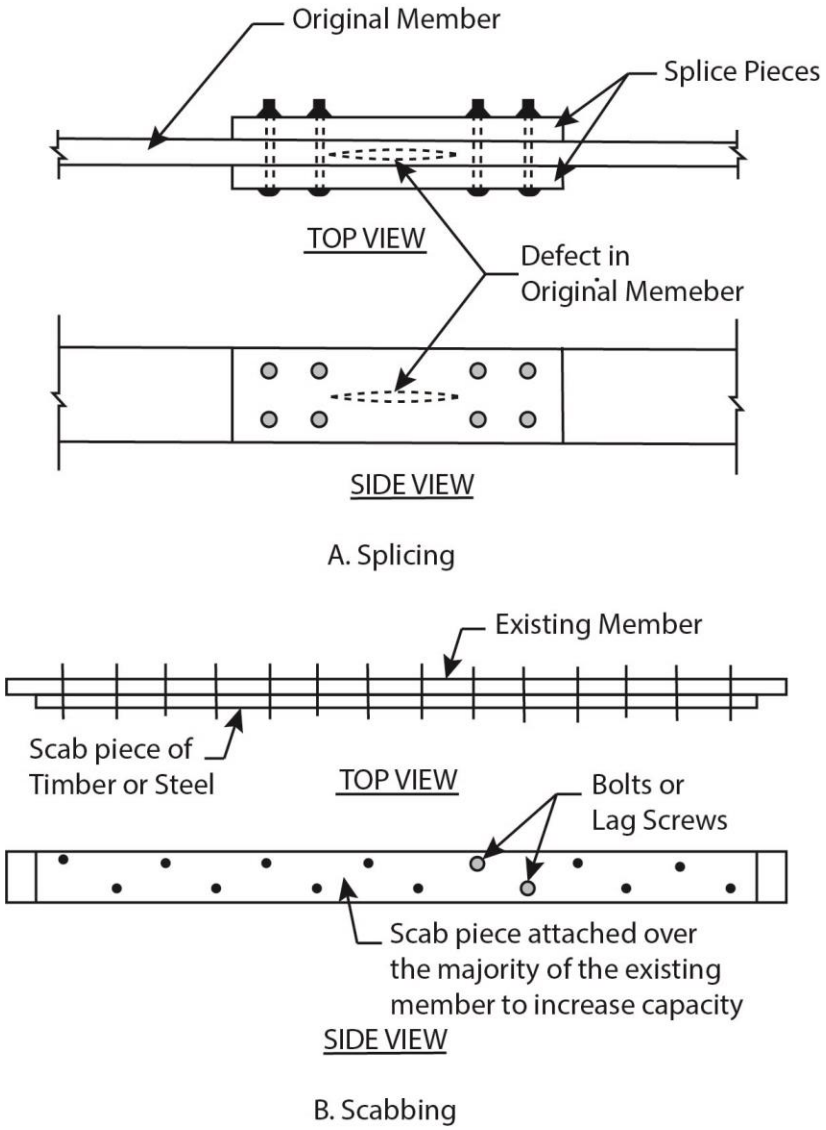


Figure 13.61 A. Splicing (top) and B. Scabbing (bottom)

13.6.3 Clamping and Stitching

Clamping and stitching are used to correct longitudinal splits that form in timber. The purpose of this maintenance operation is not to close the split or check, but rather to prevent further development by drawing the two parts together. Examples of clamping and stitching are shown in Figure 13.62.

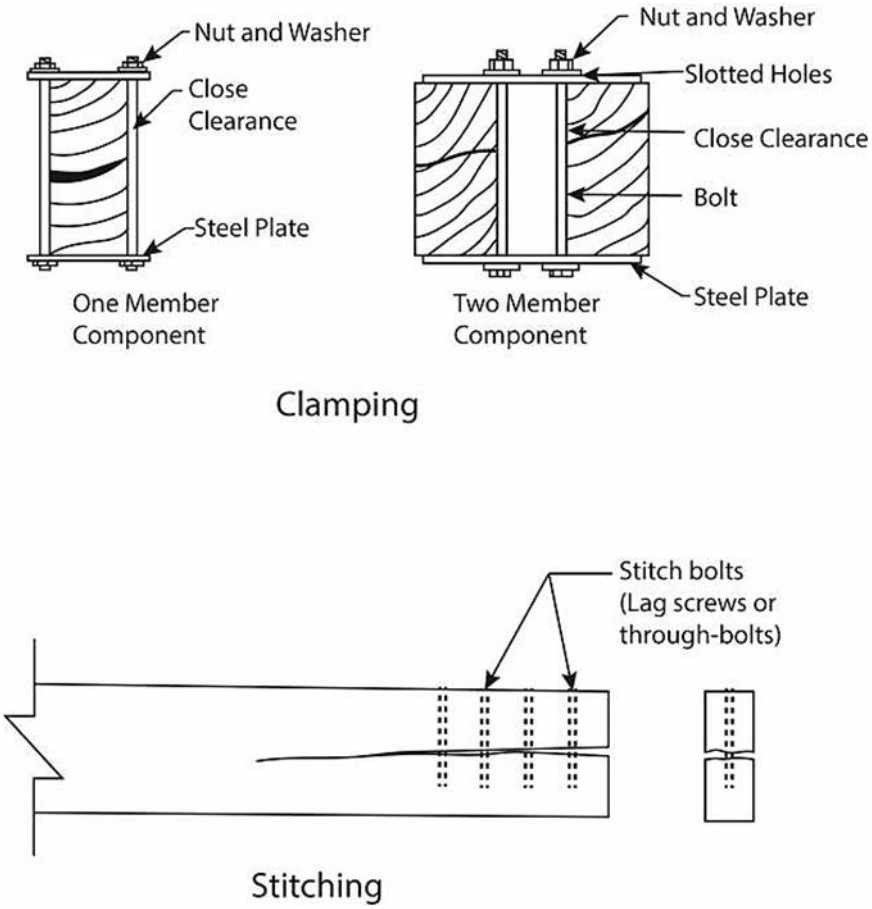


Figure 13.62 Clamping (top) and Stitching (bottom)

In clamping, plates are placed on the narrow cross sectional sides of the member. External bolts draw the plates together, clamping the member. Clamping has the advantage over stitching in that the cross section of the member is not reduced. An example of clamping is shown in Figure 13.63.

Figure 13.63 Repair to Split Timber Pile Using Steel Clamps (Courtesy of Washington DOT)

Stitching consists of placing stitch bolts which can either be lag screws or through bolts through a member spanning the split. As the screw or bolt is drawn tight, the split is drawn together.

Bolts should only be tightened to a point where they take tension. Stitch bolts should be no larger than the maximum nut size permitted in the structural grade.

13.6.4 Timber Cap Repair and Replacement

The need to increase the load capacity of a bridge may arise due to improper sizes or defects in a particular member. An example is wood pier caps that have developed large lengthwise shrinkage cracks or many splits in the vicinity of bolt fasteners. A structural analysis may indicate such caps rate significantly lower than other members because of such defects.

Provided the original cap is still in good structural condition and no decay is evident, strengthening the cap is often easier and cheaper than replacement. In some cases replacement may be very difficult due to access limitations or other factors and strengthening is the only logical alternative.

During the preliminary planning stage, necessary procedures should be determined. These procedures should be based on assurances that the existing cap and columns are in good condition and a determination the required new section will meet the needs.

In the following procedure for timber cap strengthening, the new members will help to support live load only, since the original cap remains in place to carry the dead load.

The resources which will be needed should be ascertained. They may include:

- Access to pile cap
- Light lifting equipment
- Wrenches
- Heavy duty drilling equipment
- Small hand tools

Suggested Procedure

Timber Cap Strengthening

1. Construct scaffolding, as required, around existing bent.
2. If pile diameter is wider than existing cap, notch existing piles or columns to accommodate new timber cap members.
3. Place new members snug against existing cap and stringers; temporarily clamp in place.
4. Drill bolt holes 1/16 inch larger than the bolts being used.
5. Place bolts, tighten, and remove clamps.
6. Remove scaffolding.

A schematic for timber cap strengthening is shown in Figure 13.64.

When to Call the Engineer

Call the Engineer to determine size, number, and spacing of bolts when attaching new cap strengthening timbers.

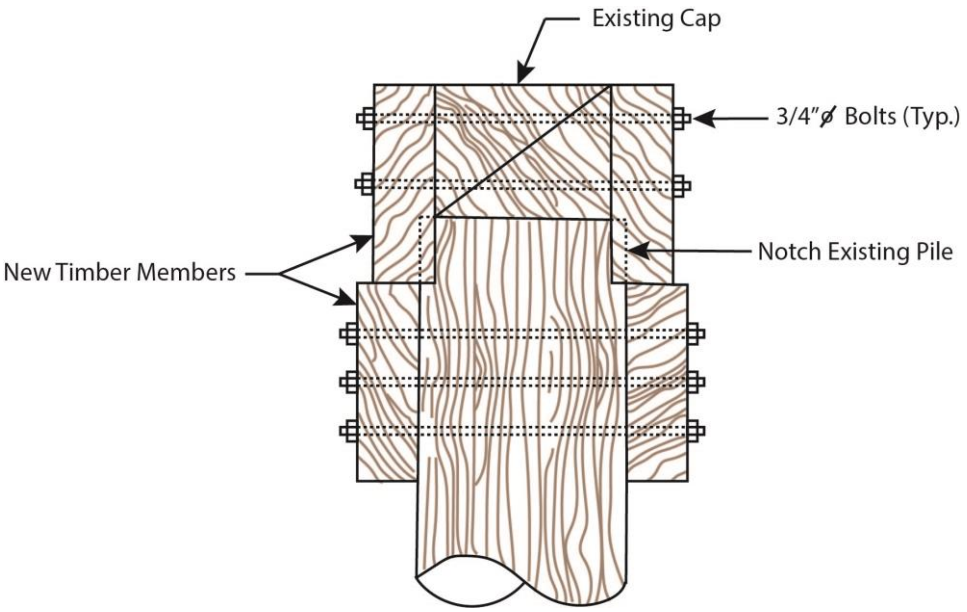


Figure 13.64 Timber Cap Strengthening

Decay on the bearing ends of timber stringers can be repaired by increasing the width of bearing provided by the cap. This decay is generally the result of dirt allowed to build up on the cap between the stringers.

Suggested Procedure

Scabbing Timber Cap to Increase Bearing Area

1. Scab a 6 inch by 12 inch to each side of an existing timber cap using split ring connectors and bolts.
2. The existing cap must be sound timber.
3. Raise the scab members into position and clamp to cap or use bolts to temporarily hold in position.
4. Drill through the scabs and the cap.
5. Remove the scab members and cut grooves for the split rings.
6. Put the scab members back into place and place bolts and washers to complete the repair.
7. The primary resistance to the imposed load is the split-rings.

An example of a scabbed timber cap with split rings is shown in Figure 13.65.

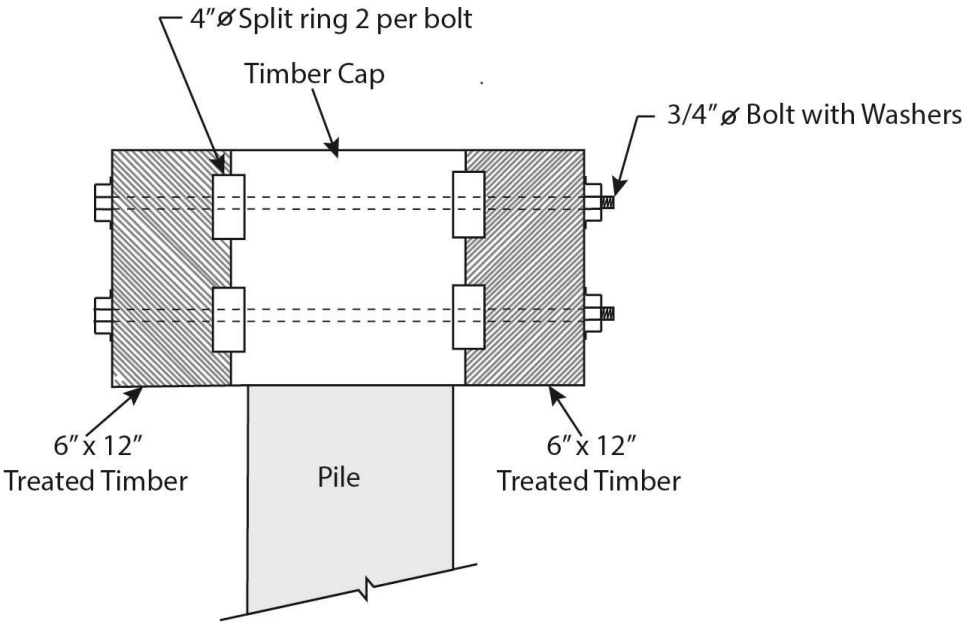


Figure 13.65 Scabbed Timber Cap with Split Rings

There are two different methods to be used for removing load from the existing cap. These are jacking from cribbing and jacking from the bent piles.

For replacement of abutment caps, place cribbing in front of the abutment, leaving room for the removal of the old cap and replacement of the new cap, as shown in Figure 13.66. For timber spans use a false cap to jack against. Spans with steel stringers jack against the bottom flange of the beams. Jack up the span about 1 to 2 inches. Cut off the drift pins connecting the cap to the piles. Slide the old cap off the piling and move the new cap into position. Lower the jacks. Connect the cap to the piles with steel straps and drift pins.

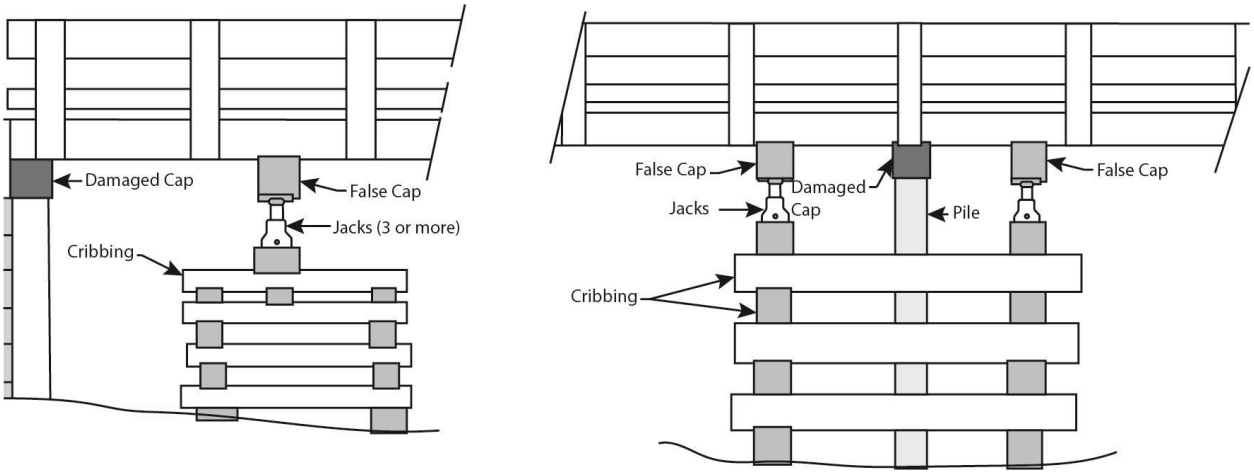


Figure 13.66 Jacking at Abutment (left) and at Pile Bents (right) using Cribbing

Timber Cap Replacement

A common maintenance problem with pile caps is decay followed by longitudinal cracks and crushing caused by the load on the cap. The pile cap must be replaced when any of these problems arise. The superstructure is jacked from both sides of the existing structure or from a

temporary bent. The decayed cap is removed and a new cap is secured in position. Figure 13.67 shows an example of jacking the superstructure so that a deteriorated timber cap can be replaced.

Figure 13.67 Timber Cap Replacement (Courtesy of Virginia DOT)

To avoid future decay, timber pile caps that have deteriorated may be replaced by steel beam caps. Stiffeners may be welded between the flanges directly over the piling. The steel cap is secured to the piling with a piece of flat stock bent to encircle the top of the pile and welded to the bottom of the cap. A schematic example of a steel replacement cap for a timber pile is shown in **Figure** 13.68.

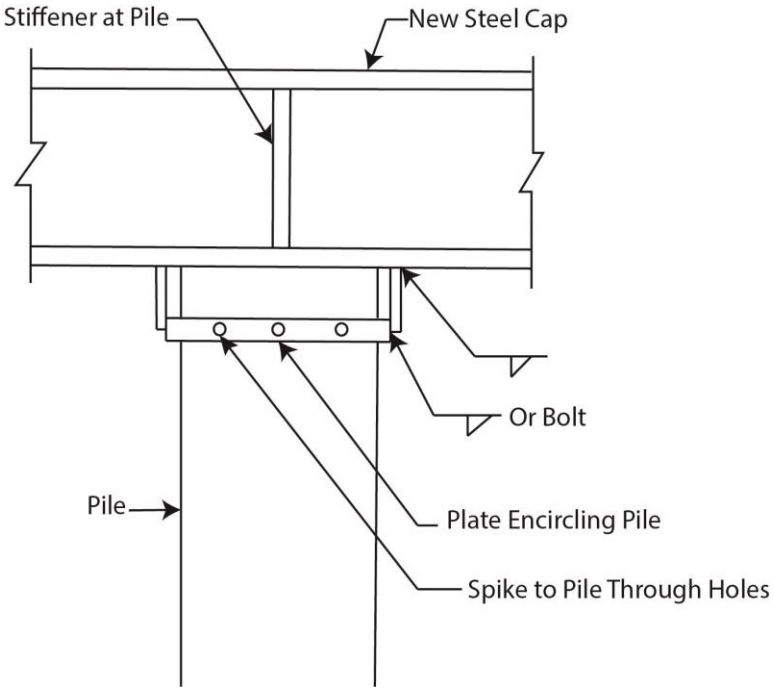


Figure 13.68 Steel Replacement Cap for Timber Pile

Figure 13.69 shows the steel cladding used to repair a deteriorated timber cap. The entire load is carried by the steel once the repair is completed. The existing cap is raised up about 1/2 inch. Steel angles are placed between the pile top and the cap. The modular sections of the cladding

are put up and bolted together at the midpoint between piles. When the members are assembled, straps are welded above and below the cap. A cross member is welded between the support angles and a vertical strap with holes predrilled is welded to the strap. Lag screws attach the strap to the timber post.

Figure 13.69 Steel Cladding Used to Repair Deteriorated Timber Cap (Courtesy of Washington DOT)

When replacing a timber cap, it is important not to overload any single pile during the jacking process. The photo in Figure 13.70 shows the use of pile jack stands used to lift the timber caps off the piles. The jack stands are clamped to the piling. The stands have the advantage of not damaging the piles and can lift a span that is over water or on a steep hillside. In the position shown the jacks are positioned so the ears that stick out from the jack stands are under the cap. Force can then be applied to raise the cap enough to either insert angles between the cap and pile for the steel cladding on the cap, or cut off the drift pins before removal of the deteriorated cap. After the drift pins are cut, the jack stands can be loosened and rotated 90 degrees and then retightened against the piles. Steel jack beams can be located up against the timber stringers (beams), and over the ears of the projections from the jack stands. Jacks can then be placed and the superstructure lifted off of the cap. The deteriorated cap can then be removed and replaced.

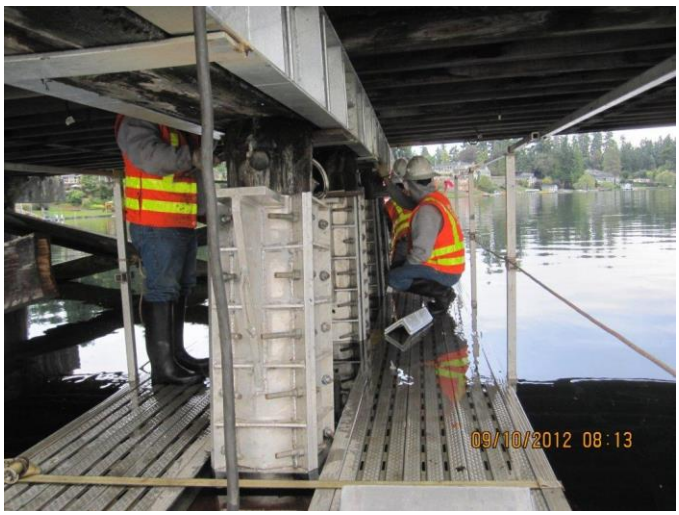


Figure 13.70 Use of Jack Stands to Ensure Adequate Load Transfer for Pile Splices (Courtesy of Washington DOT)

13.6.5 Post Splicing

The service life of deteriorated marine wood piles can be prolonged in some instances by repairing the piles. Repair methods include encasing the damaged wood pile with some type of jacket or sheeting (e.g., plastic, steel, or concrete) or removing the damaged portion and replacing it with a new piece that is spliced with the old wood pile. For example, damaged creosote-treated wood piles can be repaired using a wire-mesh reinforced shotcrete jacket, as shown Figure 13.60. Ground repair of wood poles involves screwing a metal sleeve around the base of the pole and filling the space between the sleeve and the pole with aggregates and resin as discussed earlier in Section 13.3.3.

The preliminary planning for post splicing includes:

- Determine extent of pile damage
- Design and fabricate pile steel splice sleeve
- Prepare to keep traffic off side of bridge being repaired
- Interacting with engineer as to extent of cutting and splicing

The resource requirements include:

- Timber cutting and drilling equipment
- Light lifting equipment
- Staging of traffic to reduce load over repair area

Suggested Procedure

Post-Splicing Timber Pile Repair

1. Move traffic from side of bridge being repaired.
2. Remove cross bracing as necessary to install new pile.
3. Install supplemental member to support cap.
4. Remove deteriorated portion of existing pile, cutting end off level. If the deterioration is due to rot, treat the raw end of the pile to be left in place with a fumigant. Fumigants come in a paste that can be applied to the raw wood. It would also be good to drill in and place solid fumigants according to the manufacturer specifications 6 to 8 feet along the pile to be left in place. The raw end of the pile should be covered with waterproof material after the paste fumigant is applied.
5. Install new treated timber pile and pile-splice sleeve.
6. Drive shims or other method to ensure load transfer.
7. Re-install cross bracing.
8. Restore traffic.

A schematic of timber pile repair by post-splicing is shown in Figure 13.71, and an elevation view photo is presented in Figure 13.72.

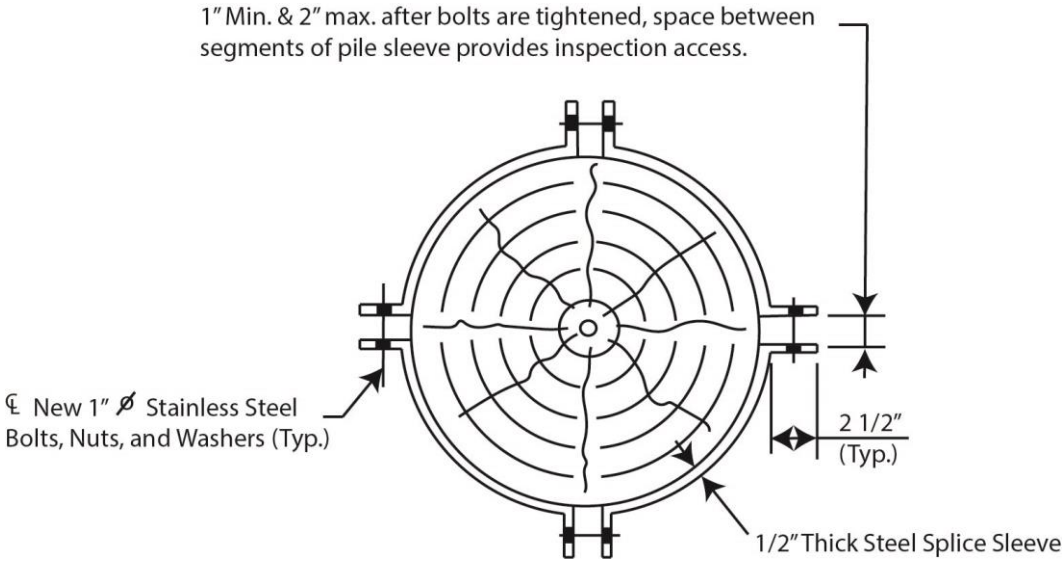

1" Min. & 2" max. after bolts are tightened, space between segments of pile sleeve provides inspection access.
℄ New 1" Ø Stainless Steel Bolts, Nuts, and Washers (Typ.)
2 1/2" (Typ.)
1/2" Thick Steel Splice Sleeve

Figure 13.71 Timber Pile Repair Schematic

Figure 13.72 Timber Pile Repair - Elevation View (Courtesy of Washington DOT)

When the portion below the ground is still sound, treated timber piles that have decayed or have been damaged by fire or impact can be repaired without the need for driving a new pile.

Suggested Procedure

Treated Timber Pile Repair Procedure (Sound Pile Below Ground)

1. After the required auxiliary support is in place, the old pile is cut off below the decayed or damaged area.
2. A new section of pile is cut about 6 inches shorter than the section removed. Plates are placed on top of the pile in place and on the bottom of the new section of piling. A drift pin can be used to keep the plate from sliding off the existing pile.
3. A 3/4-inch bolt with a nut is welded to the bottom plate and extends through a hole in the top plate. By adjusting the nut on the bolt, the new section of pile can be raised until it is securely seated against the bent cap. Care must be taken to raise the new section of piling far enough to secure the piling that receives the loading from the bent cap.
4. After the new section is in place, 1/4-inch thick angles are welded between the plates at each corner.
5. Used girder plates or flat stock are then bolted to the timber pile and extend down on the original pile about 12 inches. The top of the pile is secured to the bent cap using straps.
6. A 3 by 3 by 2 foot block of dense concrete is poured around the pile.
7. Any falsework may be removed after the concrete is poured and cured.

Timber pilings that have decayed, been weakened by insects or marine organisms, or been structurally damaged by collision or overloading may be replaced by steel columns

The following items should be considered as part of the planning process when replacing timber pilings with steel columns:

- Condition of existing piles below surface and existing cap

- Need for cofferdam to dewater work area
- Method of temporary support for superstructure during repairs
- Provisions to restrict traffic from work area during repairs

The equipment and tools needed are as follows:

- Wood cutting tools
- Light lifting equipment
- Welding and steel cutting equipment
- Wrenches and other small hand tools

Suggested Procedure

Timber Pile Replacement with Steel Column

1. Determine all cutoff points on the existing piles and the column length needed for the repair.
2. Construct temporary support for the superstructure before beginning the repair.
3. Construct cofferdam, if necessary, and dewater.
4. Excavate so that the top of the new footing will be a minimum of approximately 9 inches below the ground line.
5. Cut existing piles off so that they will project at least 12 inches into the new footing.
6. Separate the old sections of piling from the pier cap.
7. Form and pour concrete footings over the existing pile stubs.
8. Place the anchor bolts in the footing concrete prior to initial set.
9. Cut the steel columns to the proper length and weld on the base plates.
10. Provide load transfer through shimming or other methods.
11. After concrete has reached required strength, attach new steel columns to footings with nuts and washers.
12. Attach the top of the new columns to the existing pier caps with lag screws.
13. Remove all temporary supports, backfill where necessary, and remove the cofferdam if one was used.

A schematic showing replacement of timber piles with steel columns is shown in Figure 13.73.

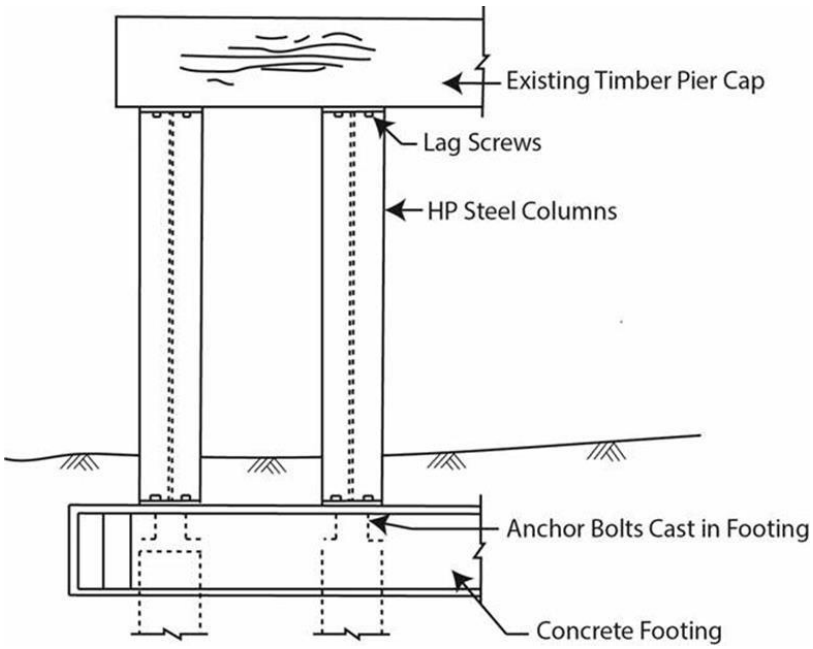


Figure 13.73 Timber Piles Replaced with Steel Columns

When using a post splice repair method a challenge can be to restore the load in the pile when a new piece is spliced into place. It can be as simple as shimming the pile to ensure adequate bearing but load transfer might not fully occur.

The construction procedure for the proper shimming between pile top and cap is not well understood by many repair crews. There is a misunderstanding that when shims are used the surfaces of the pile tops are cut to match the bevel of the wedge. This is not correct. The correct construction procedure requires the use of sets of “matching wedges”, as shown in Figure 13.74. The wedges should be cut from good structural timber such as Douglas Fir. They should be pressure treated with copper naphthenate after cutting. The repair of bridges requires some forethought to have the right parts for repair. Having several different sizes of wedges available prior to starting the bridge work is advisable.

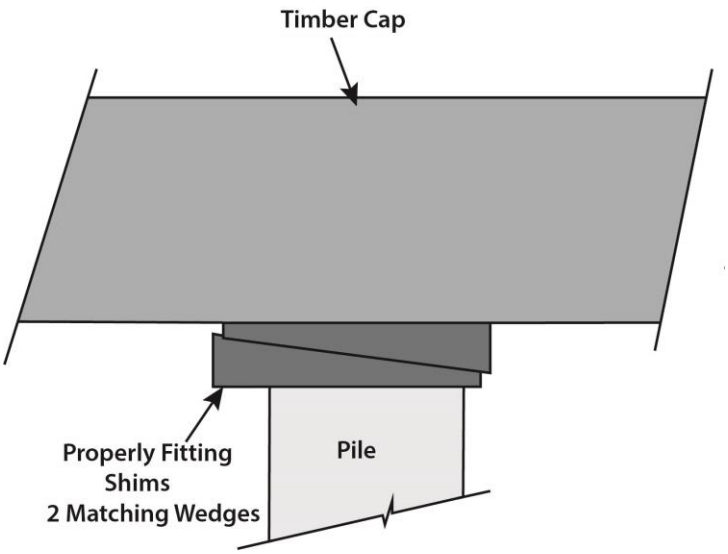


Figure 13.74 Matching Wedges for Uniform Bearing

With wedges you cannot be sure if the load has been restored to the pile or if you are overloading the pile. Wedges can also work loose over time. An alternative to wedges is jacking boxes.

Some bridge owners have developed in-place jacking boxes when replacing a timber pile with steel. Figure 13.75 shows a photo of two piles that were replaced with steel. The right pile has the jack box at the top of the pile. The left pile shows the jack box just above the pier protection. After the pile is in position a hydraulic jack is placed in the jack box. The force on the pile is then jacked to the as built pile load. The gap between the two halves of the jack box is measured and plates cut to that dimension. The plates are then welded into place. The welds and plates are then touched up with galvanic spray paint.

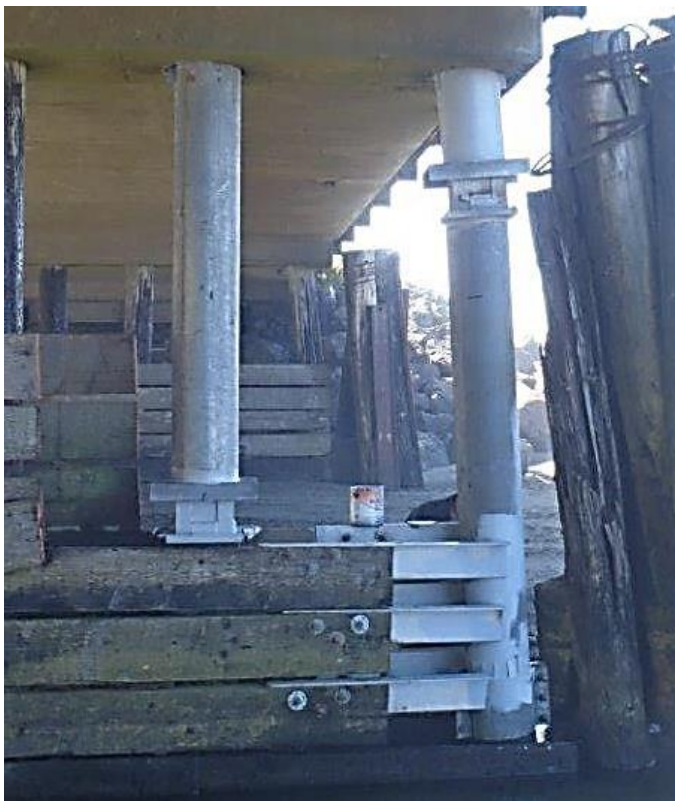


Figure 13.75 Two Steel Pile Replacements showing Alternate Jack Box Locations

The spliced pile can be loaded to design requirements, welded into place, and welded areas sprayed with a cold galvanic paint for corrosion protection.

The timber splice method previously described is not to be used when replacing piling in the abutments or end bents, because it will not provide sufficient resistance to the overturning moment produced by the force of the fill against the back wall.

Suggested Procedure

Abutment Pile Top Repair

1. Place falsework.
2. Cut the pile off below the decayed or damaged area.
3. A cylindrical steel pile shell, long enough to extend from the bottom of the cap down about 2 feet on the remaining pile, is split lengthwise and placed around the pile.
4. After the steel shell pile is pulled tight around the pile and welded, it is filled with concrete.
5. The falsework may be removed after the concrete has set.

When to Call the Engineer

Call the Engineer to determine the maximum section length that can be cut off when performing an abutment pile top repair.

Where it is not practical to repair the piling as described above, a new section of pile can be spliced to the old section using steel pipe or a band to hold the two butt ends together.

When timber pile abutments are pushed forward by the retained fill so that the pile cap is rotating, it indicates that the pile stays are broken or the abutment was constructed without stays. Abutments that are too high can also cause this problem. This defect is corrected by the following procedure:

Suggested Procedure

Installation or Repair of Pile Stays

1. Remove all soil from behind the abutment.
2. Pull the piles and caps back into position.
3. Repair or install pile stays.
4. Bury or drive deadmen behind the abutment and fasten and tighten cables, of the size specified by an engineer, to the piles with eyebolts. See "settlement repairs" above for deadman installation.

An existing substructure unit that is not capable of supporting the required load may be augmented with a timber helper bent. One example is a concrete bridge pier in which the seat damage is so deteriorated that the bearings are affected and the beams may lose bearing support. In this case, a timber helper bent adjacent to the pier would support the load and preclude bridge failure, if bearing failure did occur. The timber helper may also be used to reduce the span length and increase bridge capacity in a situation when the beams are weakened or were not designed for current loads.

When to Call the Engineer

Call the Engineer to determine size and location of the helper bent.

A structural engineer should determine the size and location of the helper bent, whether the bridge can support a pile driving rig, or if the equipment can be located off the deck. A hydraulic engineer should determine if the additional restriction is acceptable. And provisions should be made to maintain traffic safely away from work area.

The following equipment may be needed:

- Pile driving equipment

- Deck cutting equipment
- Lifting equipment
- Scaffolding, if necessary

Suggested Procedure

Helper Bent Installation

1. Locate piles so that the deck beams will not interfere with driving.
2. Cut holes in only one lane of traffic at a time.
3. For timber deck, remove sufficient amount of timber decking to permit pile to go through hole. For reinforced concrete deck, remove sufficient amount of concrete in a square pattern to permit pile to go through hole. Cut reinforcing steel at center of hole and bend back out of way of pile.
4. Set piling and drive to required bearing.
5. Cut off piling approximately 1/4 inch above bottom of existing cap. If existing cap has settled, allowance must be made for grade differential.
6. Place cover plates over deck holes, open lane to traffic, and move to adjacent lane, repeating operation.
7. After all piles have been driven and cut off, jack up superstructure approximately 1/2 inch using existing pile bent.
8. Place timber cap over both rows of piling. For end bents, only one row of piles and cap is required.
9. Lower superstructure onto new caps and strap cap to piling. Shimming may be required to obtain bearing between superstructure and timber cap.
10. Remove deck plates and reconstruct deck. If deck is reinforced concrete, splice cut bars. Replace deck in one lane at a time. Concrete deck repair may be reopened to traffic if protected by steel plates.
11. Erect cross bracing on new pile bent. For intermediate bents, cross bracing between the two new bents is also required.

An example of a helper bent installation is shown in Figure 13.76.

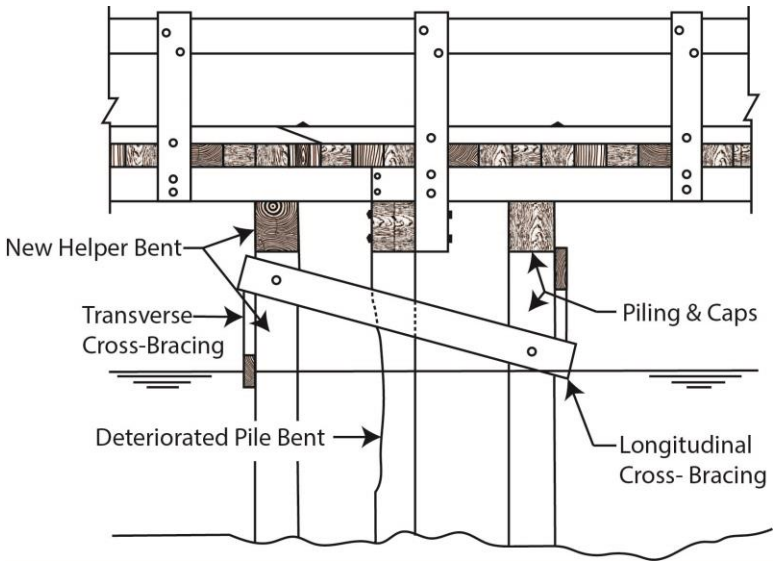


Figure 13.76 Helper Bent

Figure 13.77 shows pile driving through the deck to build a helper bent. Since piles are driven from the deck, the deck must be capable of supporting the necessary pile driving equipment. Dry land or floating pile driving rig may be used, if possible.

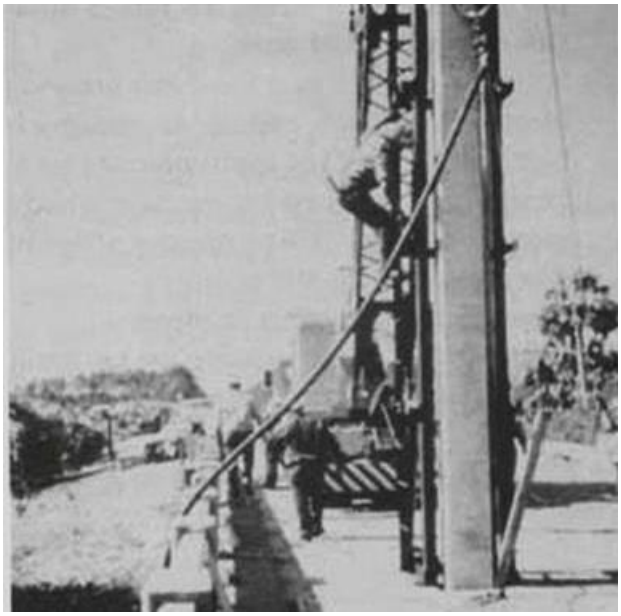


Figure 13.77 Driving Piles through Deck

13.6.6 Installation of Timber Piles

Timber piles need to be installed with special care as they are susceptible to brooming and damage. Any sudden decrease in driving resistance should be investigated since it may indicate that the pile has been split or broken. If splicing of timber piles is needed, the following guidance is recommended.

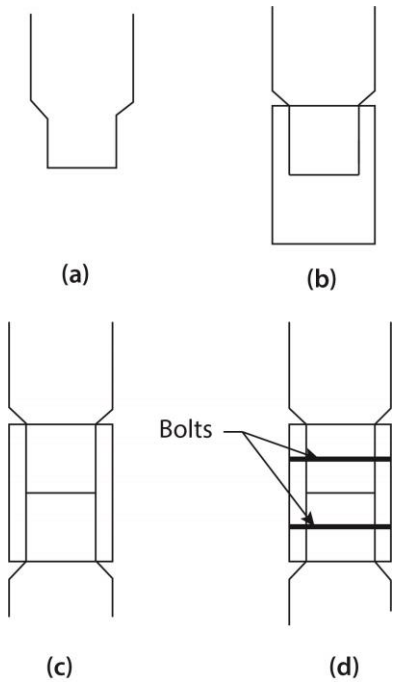


Figure 13.78 Splicing of Timber Piles

Usually, timber piles are tapered prior to splicing. The sleeve (or the pipe section) is inserted then the bottom pile is inserted. The pipe section is bolted to the piles. Sleeve joints are approximately 3 to 4 feet in length. The bending strength of the joint is much lower than the pile. Splice strength can be increased by increasing the length of the sleeve. A splice should not be installed along areas of the pile subject to high moment. This is typically at or near the ground line. An engineer can assist in calculating the bending stresses.

When to Call the Engineer

Call the Engineer to calculate bending stresses.

Precautions:

- Sleeves larger than the pile may get torn and damaged during driving and should be avoided. If this type of sleeve is to be used, the engineer should be certain that the pile sleeve is not driven through hard strata.
- Timber pile splices are extremely vulnerable to uplift (tensile) forces and should be avoided. Most timber piles are in compression, exceptions are seismic and buoyancy, or a continuous pile bent.

13.6.7 FRP Repairs

Epoxy repairs of FRP components can be classified as grouting, splicing, and pile rehabilitation. Epoxy grout is used for filling checks, splits, delaminations, and voids created by insects and decay. The epoxy prevents infiltration of water and debris. Epoxy acts to bond split sections together. Surface damage may also be repaired with epoxy gel.

Splicing involves adding pieces to span over a split or deteriorated area which are epoxy bonded in place. Epoxy is preferred over other wood adhesives because of high strength and short curing time. Epoxy bonding is not recommended for exterior use with fluctuating moisture content or for wood treated with oil type preservatives due to poor bond.

13.6.8 Pile Rehabilitation

Pile Rehabilitation can be done by two methods: pile posting and pile restoration.

13.6.8.1 Pile Posting

Pile posting involves the complete removal and replacement of a damaged section of pile with a new section of similar cross section. The new section is positioned with ¼ inch gaps at the top and the bottom and is held tightly in place with wedges. Steel pins are driven into the new pile section through bore holes in the existing pile sections. The holes are bored at a steep downward angle above each joint spaced at 90 º apart. The sides of the joint are sealed and the joint filled with epoxy. An example of pile posting is shown in Figure 13.79.

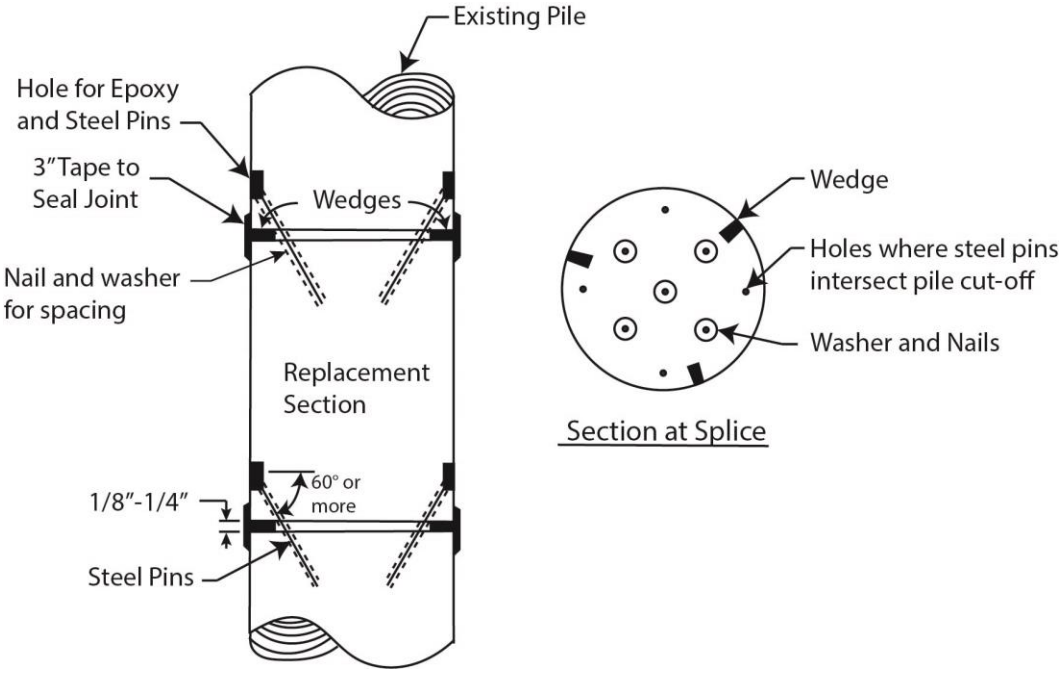


Figure 13.79 Pile Posting

13.6.8.2 Pile Restoration

Pile restoration involves the removal and replacement of a vertical wedge-shaped section of piling rather than the entire cross section. In this method the deteriorated localized section in an otherwise sound pile is removed. A similar shaped wedge is fabricated from new treated material. This replacement section is slightly smaller than the void in the existing section. Both old and new sections are covered with epoxy gel. The new section is installed and held with banding while the epoxy cures. This method is economical only where limited access prevents the use of pile posting.

The primary steps for pile wedge restoration include:

- Member preparation
- Port setting
- Joint sealing
- Epoxy injection
- Finishing

Preparation begins with removing decayed wood from a member, removing the visible decay plus an additional 2 feet of adjacent wood in the direction of the grain. This gets rid of the infecting fungi extensions. Wet wood should be dried before repairs are made. Surface that will be bonded with epoxy should be thoroughly cleaned of dirt and debris.

Ports are then installed and the epoxy is applied through one injection port at a time. As the epoxy begins to emanate through the adjacent injection port the previous port is sealed. A maximum pressure of 40 psi is recommended. Epoxy cures in a few minutes to a few hours. Injection ports may be removed and sanded or painted to meet aesthetic requirements.

The repair of wood piles with prefabricated FRP composite has a dual function of marine borer protection and structural restoration. Using prefabricated forms is environmentally friendly since no new wood preservative chemicals are introduced to the surrounding marine environment. The encasement with the FRP composite shield can reduce further leaching of chemicals from treated wood piles. Restoring structural integrity through FRP can be a cost competitive option compared to damaged pile extraction and new pile installation.

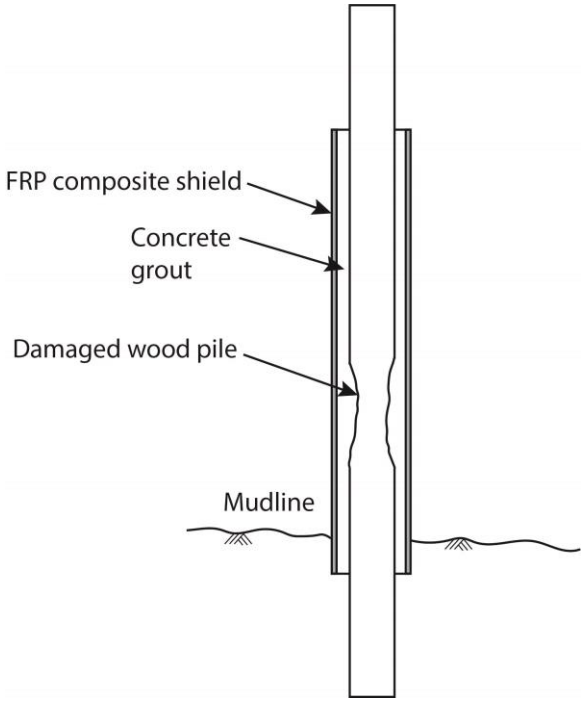


Figure 13.80 Grout-Filled FRP Composite Shield

13.7 Shoring and Temporary Supports

Shoring is normally the temporary support of structures during construction, demolition, reconstruction, etc. in order to provide the stability that will protect property as well as workers and the public. Shoring systems collect the load with headers/sheathing, deliver it into the post/struts, and then to distribute it safely into the supporting structure below. The system

must include load plates on each end to spread out the loading. A heavily loaded post can punch thru a concrete slab as well as into the ground.

Placing columns under the superstructure can require some creativity. Bridges with minimal vertical clearance do not allow enough room for cranes or other lifting equipment to work beneath the superstructure. Shoring materials should be light weight and constructed in parts, so that they can be transported and gauged to meet loading requirements. An example of lifting a support column in shown in Figure 13.81.

Figure 13.81 Lifting a Support Column

Shoring should be built as a system that has the following:

- Header beam or other elements to collect the load.
- Post or other load carrying element that has adjust ability and positive end connections. This means that the ends have a good connection to the load carrying member but there is also a way to adjust the height up or down without losing the end connections.
- Sole plate, bearing plate, cribbing, or other element to spread the load into the ground or other structure below. See Figure 13.82.
- Lateral bracing to prevent system from racking (becoming parallelogram), and prevent system from buckling (moving sideways).
- Built-in forgiveness (will give warning before failure). Example: If vertical shore is proportioned properly, (posts with length to width ratio of 25 or less) one can hear the header or sole plate crush against the post prior to the post starting to fail.

Systems used for substructure repairs are primarily intended to provide vertical support, but should have some lateral bracing for stability. This lateral stability is usually expressed as a percentage of the vertical load, typically between 2 percent minimum with 10 percent a reasonable goal. Shores need to be strong, light, portable, adjustable, and reliably support the structure. Systems should be used that are positively interconnected, laterally braced, and have slow, predictable failure mode.

When to Call the Engineer

Call the Engineer whenever shoring systems are going to be used.

A typical shoring scenario would begin with the placement of spot shores to provide initial stability, followed by individual multi post shore systems with in-plane bracing, then followed by pairs (or greater numbers) of multi post shores that are "X" braced together as two-dimensional systems.

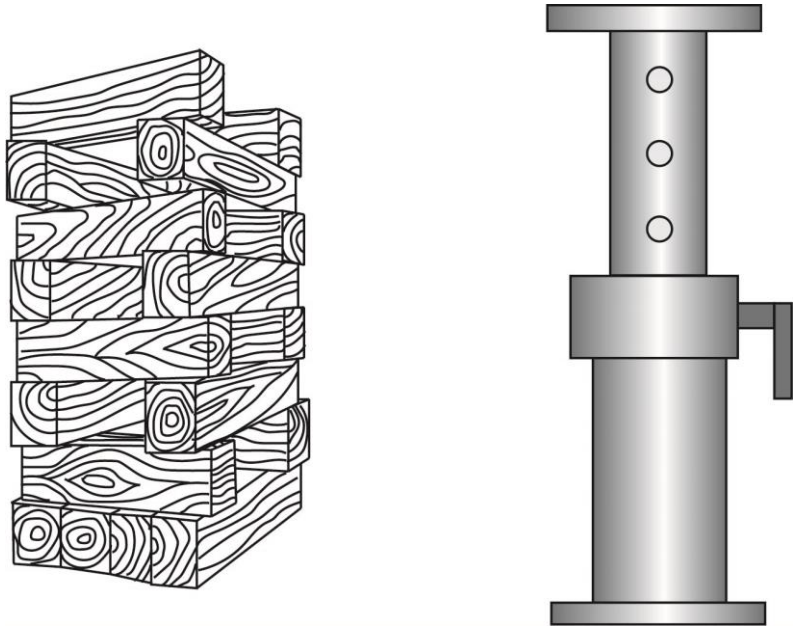


Figure 13.82 Timber Cribbing (left) and Pipe Jack (right)

Cribbing is made from composite plastic/rubber and wood and the minimum length of each piece should be 24 inches. Wood is the material of choice when constructing shoring systems because of its outstanding load-carrying capabilities and the fact that when lumbers exceeds its load-carrying capabilities it will give collapse warning signs, such as creeks, groans and visible crushing on the header and sole plates.

The height of the crib stack should not be greater than three times the width of the stack

- Minimize the overhang of the wood. It should be not be more than the shortest dimension of the wood.
- Make sure each layer lines up with its matching layer.
- Minimize the tilt of the stack. It should not exceed more than 30 degrees out of plumb.

Properly built, cribbing can support tremendous loads. A four-member crib stack built using 4x4 lumber can support 24,000 pounds. The same crib stack constructed from 6x6 lumber can support up to 60,000 pounds.

Raker shores are useful in bracing heavy walls that have cracked and/or are tilted (Figure 13.83). The shoring is installed diagonally so the wall and ground must be able to resist both

vertical and horizontal loading, and that loading may vary based on the angle of the diagonal brace. A positive connection to both the wall and ground are recommended.

The wall and ground portion of the raker can be connected through drilled-in anchors or if possible by bearing the wall plate against an existing projection in the wall surface such as a ledge. A projection can be manufactured, such as a mounted cleat, that the raker will bear against.

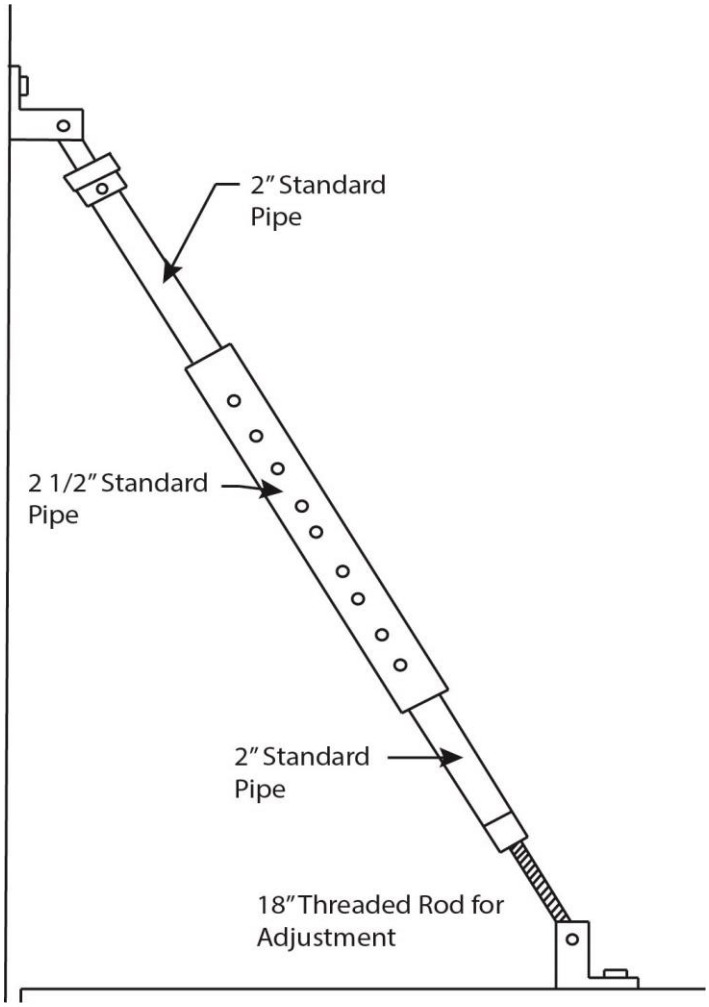


Figure 13.83 Raker Shore

Raker shores should be placed from 8 feet on center, depending on wall type and condition. They should be designed by engineers that have experience with these systems. Rakers should be built away from their installation site and carried into place. The capacity of rakers is usually limited by the cleat connections and/or the connection to the ground.

More and more bridges are relying on temporary shoring to preserve serviceability until repair funds become available. Figure 13.84 is an example of a deteriorated concrete bent cap shored up with vertical steel bracing.

Figure 13.84 Temporary Steel Shoring used to Support Deteriorated Concrete Pier Cap

13.7.1 Bridge Jacking

If the deterioration is caused by loads, or in case of extensive concrete deterioration, the superstructure may be lifted (to take the load off the substructure) prior to repair. Also, repair may require raising the superstructure to provide workspace.

Suggested Procedure

Bridge Jacking

1. Plan to remove traffic from the bridge during jacking.
2. Determine the size, number, and location of jacks that are required.
3. Ensure that jacking will not damage joints, bearing assemblies, or area supporting jacks.

The resources that will be needed should be determined. They may include:

- Jacking equipment
- Form carpentry
- Blocking to hold structure in case jacks fail
- Concrete sawing and chipping equipment
- Traffic control to move loads away from jacking area

The suggested procedure for installing a temporary bent for bridge jacking follows.

Suggested Procedure

Temporary Bent for Bridge Jacking

1. Construct a temporary bent for supporting jacks and blocking if jacking from abutment or pier elements cannot be accomplished.
2. Remove traffic from the bridge while jacking the superstructure.
3. Lift jacks in unison to prevent a concentration of stress in one area and possible damage to the superstructure.
4. If the bridge will carry traffic during repairs, restrict vehicles away from repair area as much as possible.
5. Saw cut around concrete to be removed and avoid cutting reinforcing steel.
6. Remove deteriorated concrete to horizontal and vertical planes using pneumatic breakers.
7. Add new reinforcing steel where required.
8. Apply bonding material to prepared surface that will interface with new concrete. Bonding material shall not dry out prior to casting the new concrete; otherwise, do not apply a bonding material.
9. Form as required and cast new concrete.
10. After concrete has reached required strength, remove forming, blocking, jacks and temporary supports.

13.7.2 Emergency Repairs

When the damage to a bridge is an immediate safety hazard (see example in Figure 13.85), the priorities are different than on other site visits. The objective is to assess the urgency and begin appropriate actions.

A procedure to implement an immediate closure should be developed by every bridge owner so that it can be enacted quickly when the need occurs. Crews should be familiar with their agency's policy for closing a bridge to highway and pedestrian traffic. The procedure should include contacting appropriate people within the agency, contacting police, erecting signs, establishing a detour, and contacting the media.

If the bridge damage obviously warrants closure, the procedure should be implemented. If there is a question whether the problem warrants closure of the bridge or only restricting its use, an engineer should be consulted to help make that decision.

When to Call the Engineer

Call the Engineer when debating whether to close the bridge or merely restrict use.

After the immediate safety issues are resolved, the repair urgency is determined and implemented accordingly. It may be necessary to plan the repair as the work proceeds. For example, after the type of repair is determined, the manpower and equipment is mobilized while the repair details are resolved. Special provisions for working at night and lodging the repair crew near the site may also be necessary.

Figure 13.85 Problem Bridge

13.8 Maintenance of Seismic Retrofit Components

13.8.1 Introduction

Bridges in seismically active regions may have earthquake retrofit components attached to the substructure as part of a seismic retrofit strategy. The earthquake retrofit components generally provide confinement of concrete elements, limit displacement or dissipate energy during seismic events. Seismic retrofit components are typically constructed of reinforced concrete or steel, but can use innovative materials as well such as Fiber Reinforced Polymers (FRP). Much like any other concrete or steel bridge components, seismic retrofit components require typical maintenance such as painting or spall repair defined in respective chapters of this manual. Several seismic components have special maintenance needs that will be described in this section.

13.8.2 Reinforced Concrete Confinement Shells

13.8.2.1 Overview

Confinement shells are typically used on reinforced concrete columns or pier caps where there is a lack of adequate internal reinforcement to hold the concrete in place when it cracks during a seismic event. The confinement shells prevent the pieces of concrete that may crack and break off from falling away from the member leaving inadequate compressive strength, as shown in Figure 13.86. Confinement shells are typically constructed from steel or FRP, as shown in Figure 13.87 and Figure 13.88.

Figure 13.86 Inadequately Confined Column

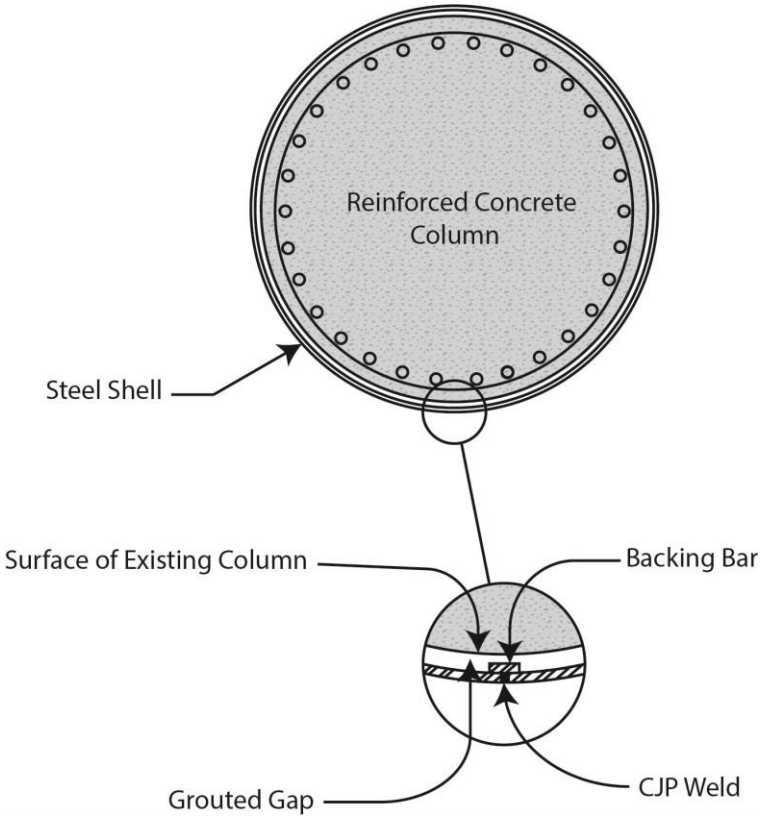


Figure 13.87 Typical Steel Column Confinement Cross Section

Figure 13.88 Fiber Reinforced Polymer Column Confinement Shells

13.8.2.2 Common Maintenance Activities

Confinement shells are fairly durable requiring only minimal maintenance. Steel shells require repainting similar to other steel components on the bridge. Steel confinement shells may have weld or base metal tears or broken bolts following seismic events. Confinement shells constructed from carbon or glass fiber wraps may be subject to deterioration from sunlight, high PH or moisture. Any signs of fiber layer delamination, broken fibers or tears in the material should be reported to an engineer for appropriate repair specifications and methods. This may include removal and replacement of the damaged FRP wrap.

13.8.3 Earthquake Restrainers

See Section 12.6.3 for the maintenance of earthquake restrainer cables.

13.8.4 Viscous Dampers

13.8.4.1 Overview

Viscous dampers are used on some structures to absorb seismic energy and to limit the forces imparted on the structure. Viscous dampers are constructed of steel and look something like large shock absorbers, as shown in Figure 13.89. The dampers are generally connected between the bottom of the superstructure and the substructure although they can be configured in other ways. Viscous dampers typically consist of a steel cylinder filled with silicone oil and a piston system that moves back and forth through the fluid during a seismic event. The piston head has space or slots on the edges that allow the heavy fluid to slowly squeeze past the piston head as it moves inside the cylinder, as shown in Figure 13.90.

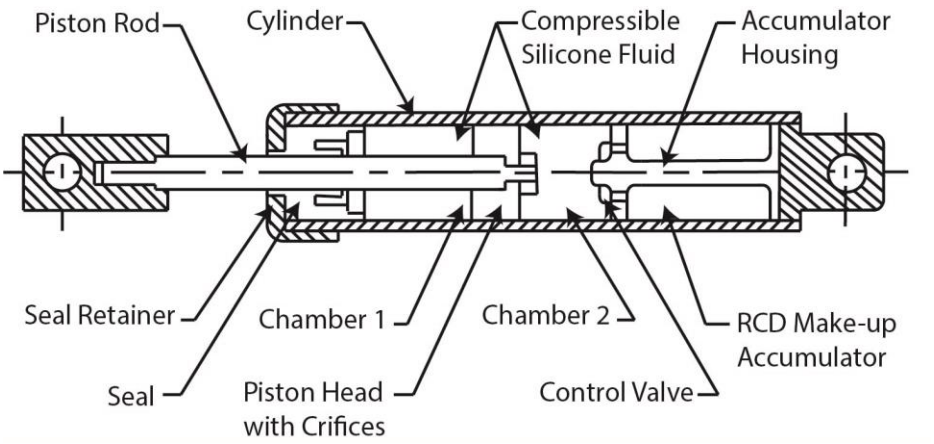


Figure 13.89 Viscous Damper Cross Section

Figure 13.90 Viscous Dampers (Courtesy of California Department of Transportation)

13.8.4.2 Common Maintenance Activities

Viscous dampers rely on the internal fluid in order to operate correctly during a seismic event. The seal at the piston end of the cylinder is known to leak in some bridge applications. The seals tend to wear out over time due to the daily thermal movement and general vibration in bridges. When the seals are leaking, the area under and around the piston end of the damper will have heavy oil splattered or dripping all over the structure and is readily visible. Some of the more modern viscous dampers have gauges on top of the cylinders to easily check the fluid levels. A small amount of fluid loss can have a significant impact on the performance of seismic dampers. Viscous dampers that have lost fluid can be refilled using the manufacture recommended fluid. Refilling the dampers in place requires a fluid pump capable of generating the required pressure inside the damper cylinder. The fluid pump is connected to the damper cylinder port and fluid is pumped in until the desired capacity has been achieved. The pumping should be done slowly as the pressure will build and dissipate as the piston moves. If the seals are leaking excessively or if the recommended pressure cannot be achieved, the damper will need to be factory resealed or replaced. Both resealing and replacement will require removal of the damper from the structure. If resealing or replacement is required, the new dampers(s) should be ordered and received before the existing dampers are removed from the structure.

Bridge dampers are likely custom made for each structure and may take several months to get replacements manufactured.

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Chapter 14 - Culverts

14.1 Purpose and Definition of a Culvert

Culverts, previously introduced in Section 3.1.5, may have a span long enough to be considered a bridge. For the purposes of this discussion, a culvert will be defined not by span length but by construction. Regardless of span, a culvert is normally designed hydraulically for fully submerged conditions (excluding bottomless) and have earth fill on top. Therefore their hydraulic and structural design methodology often differs from a typical bridge.

Some culverts have a long enough span to be classified as bridges, while others are no more than pipes under a road or driveway. The commonality is construction below and independent of the roadway surface. Culverts typically do not have a deck, superstructure, or substructure, with the exception of the bottomless culverts which have a substructure. As previously discussed in Section 3.1.5, a culvert is primarily a hydraulic structure, and its main purpose is to transport water flow efficiently. A culvert takes advantage of submergence to increase water carrying capacity. The most common materials used in the construction of culverts:

- Concrete (Pre-Cast and Cast-in-Place)
- Metal (Steel and Aluminum)
- Plastic (HDPE)
- Masonry

14.2 Components of Culverts

The nomenclature for the components of Culverts is included below.

- Apron – Area that is intended to reduce the potential for scour as water enters and exits the culvert
- Barrel – The opening that allows the water to pass under the roadway
- Crown – The top, or uppermost part, of the culvert
- End Section – An appurtenance attached to the end of a culvert to improve hydraulic efficiency, and anchorage
- Haunch – The area outside the pipe, and below the Spring line and the bottom of the pipe
- Headwall – A structure placed at the inlet or outlet of a culvert to protect the embankment slopes, anchor the culvert, and retain soil around the culvert opening
- Invert—The inside bottom of the culvert in cross section, which serves as the flow line of the culvert
- Joint - the location where two sections of material that the culvert is composed of are mated.
- Nosing—Location between culverts in a multiple culvert crossing which is often protected with concrete or riprap

- Parapet – A type of head wall above the opening of the culvert which not only retains fill, but projects above the roadway surface and acts as either a traffic barrier or a foundation for guard rail.
- Spring line — A line extending parallel to the invert along the length of the culvert at its widest point.
- Toe wall/Cutoff Wall – A full width wall which extends downward from the invert of the culvert at the inlet or outlet end to prevent piping and undermining
- Wingwall – A tapering wall the originates from the top of the culvert and is intended to retain soil under the roadway

14.3 Common Types of Culverts

14.3.1 Corrugated Metal Pipe/Pipe Arch

A corrugated metal pipe or pipe arch is typically constructed of steel or aluminum, with corrugations in the pipe to provide enhanced structural stability. Spans for preformed round pipe can extend up to approximately 12 feet in diameter, where spans of round corrugated plate structures can extend up to 50 feet. Corrugations typically vary from 2-2/3 inches x 1/2 inch to 5 inches x 1 inch. Some deep corrugation structures may have 15-1/2 inches x 5-1/2 inches or 16 inches x 6 inches corrugations.

Because corrugated metal pipes are flexible, meaning that the pipe has relatively little resistance to bending, they are highly dependent on the integrity of their surrounding backfill for their structural stability. Preformed corrugated metal pipe sections are typically delivered in 20 foot sections which require a joint every 20 feet. Joints are typically held together with bands that have indentations to engage the first one or two corrugations from each pipe at the joint, and the band is then bolted to tension it around the pipe. These type joints may be subject to leaking and misalignment if not carefully installed. Careful alignment and installation of the joint combined with adequate compaction of bedding and backfill will minimize the probability of joint problems developing.

Corrugated metal plate pipes/arches are composed of individual corrugated metal plates that are bolted together in the field to construct the culvert. These corrugated plates are often of heavier gauge metal than the pre-formed pipes, and are delivered in smaller sized pieces. Because each plate typically has multiple bolts fastening it to adjacent plates, the joints between aren’t typically misaligned, and so leak less frequently. An example of a twin corrugated metal plate pipe arch culvert is shown in Figure 14.1.

Figure 14.1 Twin Corrugated Metal Plate Pipe Arches

14.3.2 Rigid Concrete Pipe

Rigid concrete pipe is made of reinforced Portland cement concrete. The pipe sizes may range up to 15 feet in diameter, with roughly equivalent waterway openings in oval and arched pipe. Concrete pipe is typically precast and delivered in 8 foot lengths, meaning there will be a joint every 8 feet along the length of the culvert. Concrete pipe is rigid, which has a high resistance to bending, making soil stability around the pipe less critical than with metal pipe, but still important. Concrete pipe is highly dependent on the stability of the soils beneath the pipe to maintain the alignment and the integrity of the joints. Should the bedding move, the joints are likely to separate.

14.3.3 Corrugated Polyethylene Pipe

Corrugated high density polyethylene pipe is typically used for culverts with spans of up to 8 feet. Often the culverts are double walled, with a corrugated exterior wall and a smooth interior wall. The culverts are typically delivered in 20 foot sections, requiring joints every 20 feet. Joints are typically bands that engage the first corrugations adjacent to the joint or bell and spigot type joints which snap together. In rare cases, the joints may be welded.

14.3.4 Precast Concrete Box /Arch

Precast concrete box culverts can be designed for various depths of cover and live loads. Standard concrete boxes are available with spans up to 12 feet with wall thicknesses from 4 to 12 inches, where precast arches may span up to 100 feet. Because the precast sections are being transported to the construction site, the size of the sections used to construct the culvert varies to control the size and weight for transportation. Precast culverts will typically have a joint every 6 feet to 8 feet.

14.3.5 Cast-in-Place Concrete Box or Arch

Because cast in place box culverts and arches are not limited by the transportation constraints that limit precast culverts, they may have longer spans or different configurations. It is not uncommon for cast in place boxes to have multiple barrels to provide additional span or waterway openings, as shown in Figure 14.2. Cast-in-place concrete culverts and arches can be designed and constructed to meet specific site conditions.

Figure 14.2 Twin Barrel Cast-in-Place Concrete Box Culvert

14.3.6 Corrugated Metal Box

Corrugated metal box culverts may be constructed of either galvanized steel or aluminum and typically range in span from 9 feet to 25 feet. Vertical openings range from 3 feet to 13 feet. They are usually constructed of corrugated structural plate with the corrugation size depending on the span and configuration of the box.

14.3.7 Masonry

Masonry culverts are typically constructed of stone, brick or concrete block, an example of which is shown in Figure 14.3. This type of culvert was commonly built prior to the 1920's because these relatively durable materials were readily available for construction. The masonry units may be held together with mortar, or they may be dry (no mortar). Dry masonry relies on the friction/compression of well fit masonry units to hold the culvert together. Since the 1920's, the use of masonry in culvert applications has largely been limited to headwalls.

Figure 14.3 Masonry Culvert

14.3.8 Bottomless Pre-Cast Concrete Box or Arch

Precast 3-sided concrete culverts or arches can be designed for various depths of cover and live loads. Unlike other culverts, the natural stream bottom serves as the invert, and the legs of the box or arch are founded on concrete footings. Spans can be up to 100 feet for arches. As with the other precast culverts, size and weight for transportation will often govern the size of the

precast pieces and the number of joints. An example of a bottomless pre-cast arch is shown in Figure 14.4.

Figure 14.4 Bottomless Precast Concrete Arch

14.3.9 Bottomless Cast-in-Place Concrete Box or Arch

Because bottomless cast in place box culverts and arches are not limited by the transportation constraints that limit precast culverts, they may have longer spans or different configurations. Like their precast counterparts, cast-in-place bottomless structures are founded on concrete footings, and the invert is the natural stream bottom. Cast-in-place concrete culverts and arches can be designed and constructed to meet specific site conditions.

14.3.10 Bottomless Corrugated Metal Box or Arch

Bottomless corrugated metal box culverts and arches may be constructed of either galvanized steel or aluminum and typically range in span up to 40 feet. Unlike the corrugated metal structures with a bottom, the legs of a bottomless corrugated metal box or arch are founded on concrete footings, and the invert is the natural stream bed. They are usually constructed of corrugated structural plate, with the corrugation size depending on the span and configuration of the box. An example of a bottomless corrugated metal arch culvert is shown in Figure 14.5.

Figure 14.5 Bottomless Corrugated Metal Arch Culvert

14.3.11 Bottomless Masonry Culvert (Masonry Arch)

Bottomless Masonry Culverts (masonry arches) are typically constructed of stone or brick, and were more commonly built the prior to the 1920's. Stone and brick are relatively durable

materials that were readily available for construction. Similar to the previously discussed bottomless culvert structures, the bottomless masonry culverts also used the natural stream bottom as the invert and were founded on either concrete or masonry footings. More recently, the use of masonry in culvert applications has largely been limited to headwalls.

14.4 Culvert Defects

14.4.1 Debris

Debris inside culverts, as well as at inlet and outlet ends, can degrade the hydraulic capacity and performance of culverts. An example of debris and silt buildup in a corrugated culvert is shown in Figure 14.6. Accumulations of debris can re-direct the flow of water causing scour of the stream banks, road embankment, or stream bed. Debris can also cause standing water in culverts, or conditions where material stays in contact with the culvert keeping it wet, and accelerating deterioration of the culvert materials. This is of particular concern in applications with galvanized steel. The debris may be large such as trees, limbs, woody vegetation, tires, shopping carts, or naturally occurring objects such as cobbles and boulders that have been washed down stream into or around the culvert. The debris may also be smaller grained soils, such as gravel, sand or silt. Vegetation growing at the inlet may also restrict the waterway opening.

Figure 14.6 Debris and Silt in Corrugated Culvert

14.4.2 Siltation

Similar to debris, siltation inside culverts, as well as at inlet and outlet ends, can degrade the hydraulic capacity and performance of culverts, as shown in the examples in Figure 14.6 and Figure 14.7. Accumulations of silt can re-direct the flow of water causing scour of the stream banks, road embankment, or stream bed. Silt can also cause standing water in culverts or conditions where material stays in contact with the culvert keeping it wet. These situations can accelerate deterioration of the culvert materials. This is of particular concern in applications with galvanized steel. The siltation may be caused, or at least exacerbated, by larger debris, such as trees, limbs, cobbles and boulders, or vegetation. Removal of larger debris should be done in conjunction with siltation removal to reduce the potential for deposition of more silt.

Figure 14.7 Silt and Vegetation Blocking Corrugated Metal Culvert

14.4.3 Scour

Scour is the lowering of the stream bed due to the removal and transportation of stream bed material by flowing water. Scour can occur at the inlet end, outlet end, or both ends of a culvert. Scour holes may form near the culvert inlet due to the flow striking against the upstream embankment prior to entering the culvert, or as the flow accelerates upon entering the culvert if the culvert is forming a constriction in the stream. Local scour at the outlet may occur when high velocity water exits the culvert.

The stream may also be subject to general scour. General scour is the gradual degrading or lowering of the stream bed over all, or part of its length. General scour can be a concern if the area of degradation is migrating upstream toward the outlet of the culvert. This can ultimately lead to the culvert being undermined. Conversely, general scour upstream of the culvert can subject the culvert to siltation and debris accumulation.

14.4.4 Leaking Joints

Joints serve several purposes, including:

- Maintaining the flow of water through the culvert
- Keeping soil, and water from infiltrating into the culvert
- Maintaining alignment
- Preventing the sections from pulling apart

Evidence of poor joint performance includes:

- Separated or misaligned joints
- Evidence of water infiltration or exfiltration
- Soil infiltration
- Seepage at joints
- Open joints
- Sink holes in the embankment/pavement surface over the joints

14.4.5 Corrugated Metal Culvert Defects

14.4.5.1 Damaged Coatings

Steel culverts are typically coated to protect the steel from corrosion. The coatings routinely used include galvanizing (zinc) or aluminizing (aluminum). In addition, galvanized pipe can be coated with polymer or asphalt to extend its life. In all cases, damage to the coatings will accelerate the onset of corrosion at the damaged coating location, which will reduce the lifespan of the culvert as corrosion advances.

Aluminum alloy culverts are typically uncoated, because aluminum alloys develop a protective oxide barrier layer when installed in environments where pH is between 4.0 and 9.0. For comparison purposes, distilled water has a pH of 7.0, baking soda has a pH of approximately 9, and tomatoes have a pH of around 4.

14.4.5.2 Dents and Localized Damage

Pipe wall damage such as dents, bulges, cracks, tears, and creases can occur during installation and backfill of culvert. If the damage is extensive the structural integrity of the culvert may be compromised, requiring rehabilitation or replacement. In many cases, localized damaged is not critical to the long term durability of the culvert.

14.4.5.3 Corrosion

Corrosion occurs in soils or waters that have corrosive properties (high or low pH) or an abrasive environment (where abrasion removes the protective coatings on the culvert increasing vulnerability to corrosion). Acidity or Alkalinity can occur from pollutants, such as acid rain, but more typically it is due to the underlying geology of the area. Corrosion can lead to section loss, perforation, and ultimately structural failure. Some agencies may choose to utilize culvert materials of heavier gauge, or with additional coatings to obtain the necessary design life given a more corrosive environment. However, when corrosion progresses to the point where the culvert is failing (leaks, perforations, structural deformations) repairs, rehabilitation, or replacement options should be explored. During selection of the repair option, the pH of the environment should be considered. Examples of culvert corrosion are shown in Figure 14.8 and Figure 14.9.

Figure 14.8 Corrosion in Plate Arch

Figure 14.9 Corrosion Failure of Culvert Invert

14.4.6 Concrete Culvert Defects

14.4.6.1 Cracks

Cracks in concrete culverts may occur due to a number of factors. The contributing factors can often be identified by the type of crack in the concrete culvert. Types of cracks occurring in concrete culverts include:

- Circumferential cracks near joints of round concrete culverts. These cracks usually indicate that there has either been some movement or settlement of the pipe sections, the gaskets were improperly installed, or the pipe was damaged during installation.
- Longitudinal Cracking of round culverts at the invert, crown and spring line .These cracks are usually flexural cracks caused by loss of side support (Figure 14.10 left).
- Longitudinal Cracking of round or pipe arch culverts parallel to the invert. These cracks are usually shear cracks due to loss of support at the haunches (Figure 14.10 middle).
- An example of good side support is shown in Figure 14.10 (right).

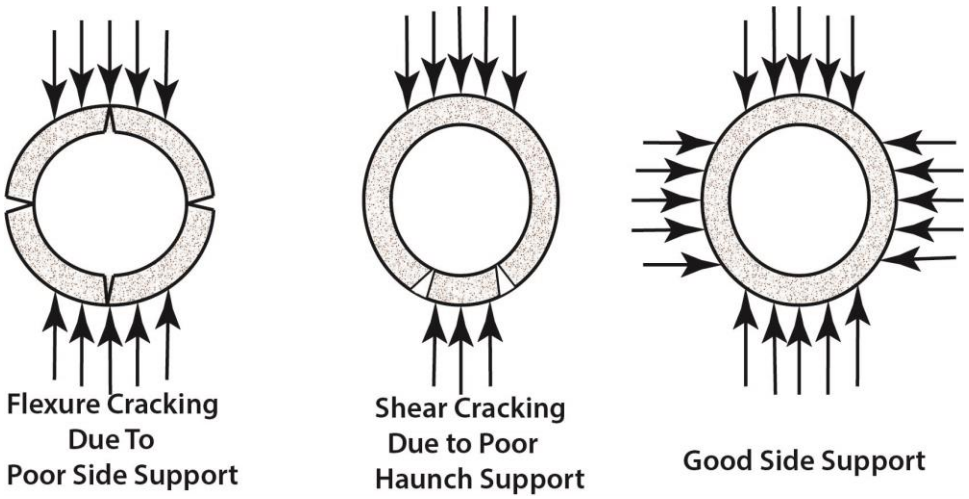

Figure 14.10 Longitudinal Cracks in Concrete Pipe

Transverse, or circumferential, cracks in round or pipe arch culverts often indicate that the pipe is in flexure. Cracks in the bottom of the pipe indicate a loss of support at midsection, whereas cracks in the top of the pipe indicate a point load was applied to the pipe at or near the location of the crack.

Cracks in box culvert and arch concrete can indicate differential settlement in the case of transverse cracks in the top, or invert and differential earth pressures in the case of the walls. Longitudinal cracks in box culverts that are not a result of reinforcing corrosion can often be attributed to flexural or shear distress in the concrete.

14.4.6.2 Spalls

A spall is a fracture of the concrete parallel to or inclined to the surface of the concrete. Spalls often occur along the edges of cracks, near joints and corners, and above reinforcing steel that has corroded.

14.4.7 Polyethylene Culvert Defects

14.4.7.1 Cracks

Polyethylene culvert pipes can sometimes develop cracks due to improper installation procedures, impacts, or stresses. Large cracks could compromise the structural integrity of the pipe, or cause soil infiltration, while small cracks may not be severe enough to need repair.

14.4.7.2 Minor Damage

Minor damage may be smaller than the width of one corrugation. Minor damage that perforates the pipe wall may cause infiltration/exfiltration of water as well as infiltration of soil. If left unchecked, this could lead to loss of soil support for the pipe.

14.4.8 Masonry Culvert Defects

14.4.8.1 Cracks

Individual bricks, stones, or blocks may be cracked, broken, crushed, or missing. Sometimes a seam or crack will follow a mortar line through a masonry unit, which then opens up a seam or crack in the individual unit. Cracks may also develop parallel to the surface of the masonry unit resulting in spalls or loss of section. Cracks may also be caused by thermal movement, or moisture related causes such as water infiltration. Cracks may also indicate that the culvert is subject to differential loading or stresses, or the loss of foundation support.

14.4.8.2 Loss of Mortar

In most masonry construction, mortar is used to cement the masonry units together. Loss of mortar may lead to displacement or loss of masonry units which can compromise the structural integrity of the culvert (Figure 14.11). Loss of mortar may be caused by moisture, thermal expansion and contraction, or loss of soil support. Loss of mortar may lead to water and soil infiltration.

Figure 14.11 Mortar Loss in Bottomless Masonry Arch

14.4.8.3 Soil and Water Infiltration

Cracking or loss of mortar may lead to water and soil infiltrating into the culvert. This can lead to the loss of soil covering the culvert, distresses in the surfaces above, and potential structural failure of the culvert from the loss of uniform soil cover. Loss of mortar may be caused by moisture, thermal expansion and contraction, or loss of soil support. In cold climates, water infiltration can also lead to freeze thaw damage to the mortar and masonry units.

14.4.8.4 Frost Action

Distress due to frost action is may take the form of uniform or differential heaving of the culvert, deformation of the culvert shape, or displacement of masonry units. After the frozen soils surrounding the culvert thaw, additional damage may occur resulting from loss of soil support, or differential loading from the soils.

In order for frost action occur, the environment around the culvert must be subject to freezing temperatures, frost susceptible soils, and a source of water. Because there is no effective way to control the temperature, and culverts are often crossing waterways, the feasible solution to prevent frost action is typically to bed and backfill the culvert on granular soils that are not frost susceptible.

14.4.8.5 Loss of Foundation Support

Like any other type of culvert construction, support from the surrounding soil is important to the proper structural performance of the culvert. Loss of soil support may be caused by weak soils, inadequate compaction of bedding or back fill, erosion or scour, moisture infiltration or frost action. Indications of foundation support loss include cracking, deformation, loss of mortar, settlement, or uplift.

For additional information about maintenance and repair of masonry structures, see Chapter 16.

14.4.9 Bottomless Culvert Defects

14.4.9.1 Scour/Undermining

Solid foundations are essential to the continued performance of bottomless culverts. Unlike culverts with bottoms, where scour begins at the inlet and outlet ends of the culvert, scour (or undermining) can begin at any point along the footings of a bottomless culvert and extend or migrate from there. Loss of support for the footings can lead to footing failure and, in turn, overall structural failure of the culvert.

14.4.9.3 Section Loss/ Spalls in Substructure

Loss of section in the footings of bottomless culverts can lead to loss of support for the structural elements of the culverts. An example of footing section loss in a bottomless culvert is presented in Figure 14.12.

Figure 14.12 Footing Section Loss in Bottomless Culvert

When to Call the Engineer

The Engineer should be notified if loss of section in footing or loss of bottom support in culvert appears to be causing distress such as cracks, bending deformation, etc.

14.5 Preventive and Basic Maintenance of Culverts

In this section, preventive maintenance refers to the on-going scheduled activities aimed at maintaining and extending the service life of culverts. Scheduled maintenance activities become important to proper culverts performance. Many agencies have a culvert inspection program, which assesses conditions and identifies maintenance and repair needs. The inspection cycle for culverts is based on the agency, but can range from 1 to 10 years. The level of detail can vary from inspection-to-inspection and can also vary within various agencies. Often culverts are checked for debris or scour issues annually, or after major storm events, whereas in-depth structural inspections happen less frequently.

14.5.1 Removal of Debris

Before beginning any debris removal work, it is essential to determine the environmental regulations that govern the water body that the culvert is carrying. The debris removal operations should be performed in compliance with local environmental regulations, such that:

- Debris is not displaced downstream where it could degrade water quality.
- A debris problem at another structure is not created.
- The spread of an invasive species is not accelerated.
- Removal of debris, such as a beaver dam, does not cause a sudden release of impounded water that would cause damage downstream.

14.5.1.1 Debris Removal

Removal of large debris at the inlet and outlet ends of a culvert is often most effectively performed with an excavator or other large equipment. Removal of large debris inside the culvert may require the use of cables, mandrels, or manned entry. If the debris blocking the entrance or exit of the culvert is a beaver dam, the appropriate permit from the state environmental agency may be required.

14.5.1.2 Silt Removal

Several methods may be employed for silt removal, included vacuuming with a vacuum truck, flushing with a water or sewer jet, flushing with a fire hose, or collection with a bucket line. Details of these methods are discussed below.

Vacuum Truck- A vacuum may be used to remove sediment and relatively small diameter debris from culverts including stones, leaves, bricks, litter and sediment. The vacuum apparatus is typically truck mounted with a holding tank ranging in size from 200 to 3500 gallons. The vacuum hose diameters range from 4 inches to 10 inches and are equipped with a metal end for breaking up sediment.

Water jet/Sewer Jet – A water or sewer jet can be either a truck or trailer mounted piece of equipment used to flush sediment and debris from culverts. The equipment consists of a high pressure pump and water supply. A hose with a nozzle is inserted into the culvert, using the high pressure water to dislodge debris and wash it to the downstream outlet of the culvert, where it is collected and removed. Nozzle configurations and water pressures can vary depending on the type of equipment used and the type of debris and culvert being cleaned. The maintenance staff should exercise caution using high pressure water nozzles to remove silt and debris in culverts with open joints, or masonry culverts with loose masonry or open joints. In these instances, the high pressure water may remove even more material from existing open joints.

Fire Hose Flushing – This equipment consists of various types of nozzles that can be placed on the end of a fire hose that is drawn through a culvert to flush or clean it. Silt/Debris is flushed to the downstream outlet, where it is collected and removed.

Bucket Line – This equipment is a cable that pulls a clam-shell-type bucket through a larger diameter culvert (greater than 48 inches). The bucket collects debris and removes it from the

culvert. The full bucket is typically pulled through the culvert in the direction of flow, and is removed and emptied at the downstream end.

14.5.1.3 Vegetation Removal

The condition of the vegetation and surroundings should be evaluated to determine the appropriate method and degree of vegetation removal. Depending on the type of channel and the conditions, the vegetation removal may be as simple as mowing the areas adjacent to the inlet or outlets to keep them clear, without increasing erosion potential.

In areas where fast growing vegetation is a concern, or obstruction of the culvert with vegetation has been a recurring problem, mechanical removal (excavation of the roots) or use of herbicides may be appropriate. Use of herbicides will be governed by local environmental regulations.

14.5.2 Scour Repair

Local scour occurs at the inlet and outlet ends of culverts due to changes in water velocity as water enters and exits the culvert. The strategies for repair vary depending on the severity of the scour, the likelihood of continued scour, and the risk of damage caused by scour.

In the case of a scour hole at the outlet of a culvert which is not threatening the culvert, its appurtenances, or adjacent properties, no action may be warranted.

In the case of scour holes that could potentially threaten the culvert, filling the scour hole with riprap (i.e., stone fill) of the appropriate size based on stream flows and velocities is an appropriate option. An example drawing for scour repair using riprap is shown in Figure 14.13. See Chapter 15 for additional information about maintaining the waterway.

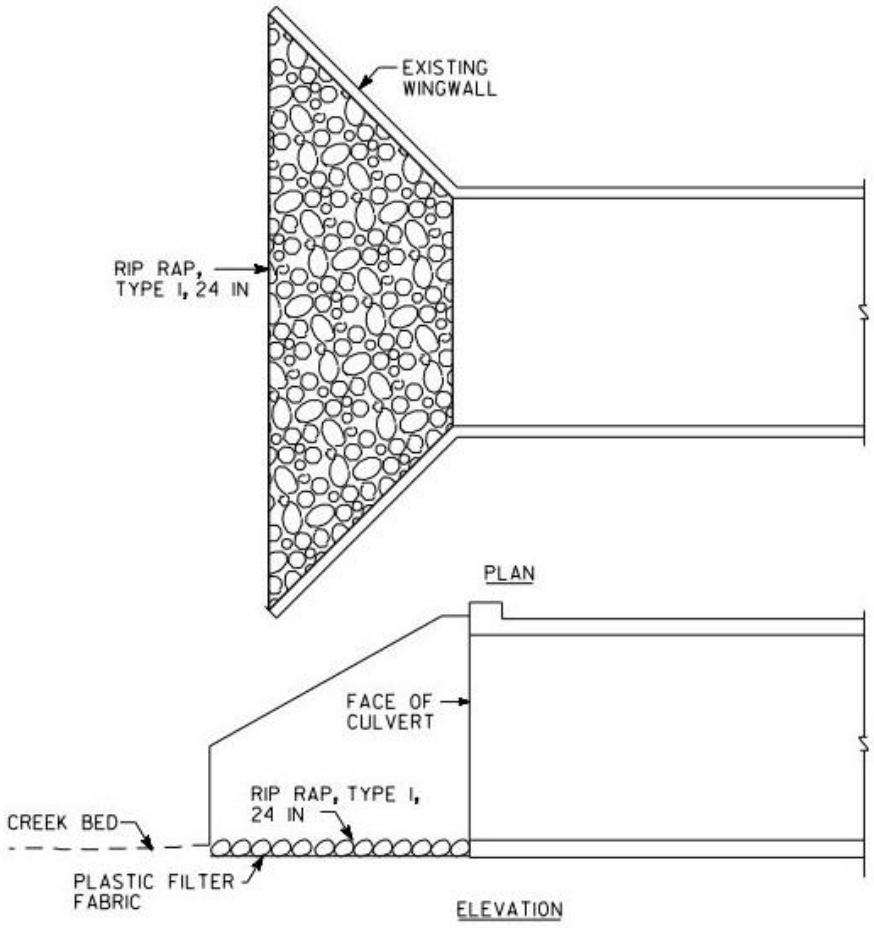

Figure 14.13 Riprap at Culvert Entrance/Exit for Scour Repair (Courtesy of GADOT)

In cases where scour has advanced underneath the culvert, a repair will not only need to be made to the scour hole outside the culvert, but also to the undermined area. One method of repairing the undermining uses self-leveling concrete (flowable fill). An example of this technique is shown below in Figure 14.14, with cross section views shown (left), plan views shown (right), existing condition(top) and final condition (bottom).

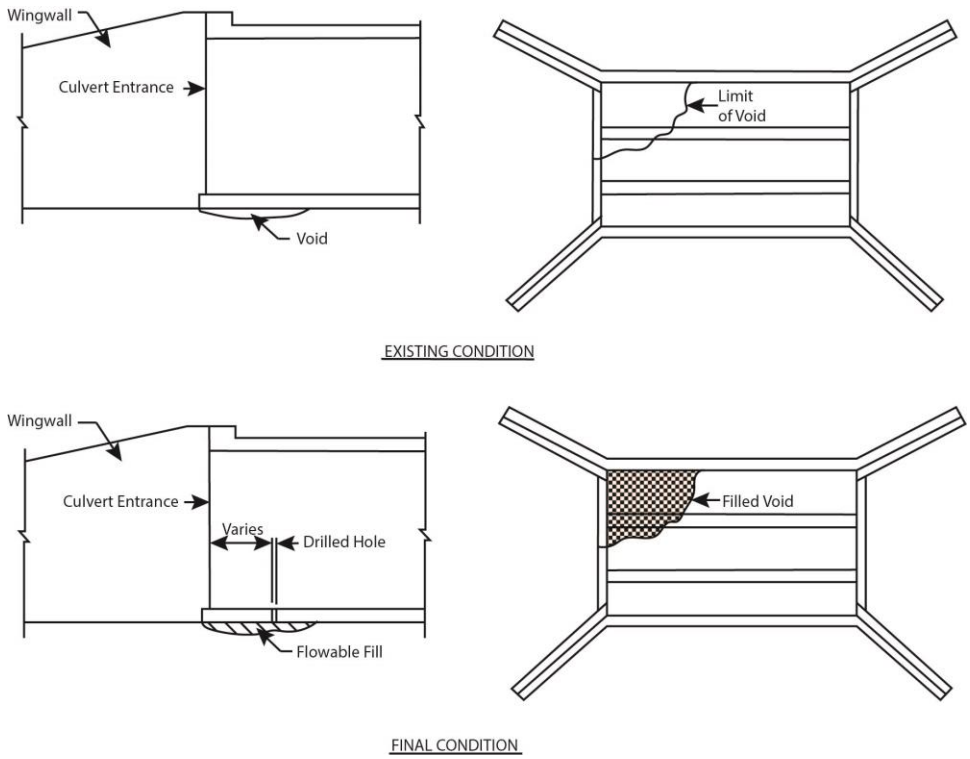

Figure 14.14 Void Repair.

It may be necessary to perform this repair technique in conjunction with either the construction of a cutoff wall, the placement of riprap, or both.

Grout bags can also be used for foundation undermining repair. For more information, see 15.2.3 Grout Bag Placement.

14.6 Repair and Rehabilitation of Culverts

A bridge maintenance crew will often be called upon to repair and rehabilitate culverts. This will go beyond simply cleaning debris and patching damaged areas.

14.6.1 Repair of Damaged Invert

14.6.1.1 Concrete Paved Invert

When the invert of a culvert has deteriorated, and the culvert is large enough for workers to enter the culvert, a new concrete invert may be cast in the culvert. Prior to relining, or as part of the new invert installation, any voids beneath the invert should be filled with concrete. An example of a repair of a damaged invert by using a concrete paved invert is shown in Figure 14.15.

Figure 14.15 Concrete Paved Invert

14.6.1.2 Saddle Plates

When corrugated metal culverts have deterioration limited to the invert or other isolated areas, another method of repair is to attach new plates of the same corrugation spacing to the invert or other location in need of repair. Prior to placing the new plates, any perforations or voids beneath the invert should be filled.

14.6.2 Relining Culverts

In the relining cases below, the result is a new structural interior to the existing pipe.

14.6.2.1 Slip Lining

By this method, the pipe is slip lined by pulling sections of the liner pipe through the existing pipe and then grouting the annular space between the existing pipe and the liner in accordance with the liner pipe manufacturer's recommendation.

When to Call the Engineer

Call an engineer to determine if the reduced waterway area will have adequate capacity to pass the design flow.

One concern with slip lining is the reduction of hydraulic opening, and therefore hydraulic capacity. Prior to undertaking a lining project, an engineer should be consulted to determine if the reduced waterway area of the lined culvert will have adequate capacity to pass the design flow. If the capacity of the lined pipe will be inadequate, other rehab/repair options should be explored.

Slip lining a culvert requires sections of liner pipe to be staged adjacent to the inlet or outlet of the culvert prior to insertion into the existing culvert. With liner pipe sections routinely being 20 feet or longer, the work area could extend off the right of way. Prior to selection of lining as

a repair option, the limits of the right of way should be established, and a determination made if the proposed liner pipes will cause encroachment on adjoining lands during installation. If encroachment is likely to occur, options such as temporary or permanent right of way acquisition or working with the supplier reduce the length of the liner pipes should be explored.

A variety of different pipe materials are used for slip lining. The process of installing each of them is essentially the same with some minor variations based on the material. Some materials that are commonly used to slip line culverts are:

- Corrugated Metal Pipe
- Corrugated Metal Plate Pipe/Plate Arch
- Smooth Interior Metal Pipe
- High Density Polyethylene Pipe

In cases the where inlet or outlet are not controlling the hydraulic capacity, the installation of a liner with a smoother interior than the original culvert can mitigate the reduction capacity caused by the smaller waterway opening, however it may increase water velocities at the culvert outlet. Some examples of smooth interior pipe include HDPE, smooth metal, Spin Cast and Cured in Place Pipe. Corrugated metal pipes are not smooth and should not be used for this purpose.

Figure 14.16 shows the steps involved to line a corrugated metal plate pipe arch. If the existing footing is adequate, it can be used as the footing for the liner arch. If not, repairs to the existing footing will be necessary as described in Section 14.6.6 Footing Repair of Bottomless Culverts. In the case of a round pipe, perforations in the invert and voids below it must be filled, and blocking beneath the invert of the liner pipe is typically provided to maintain an annular space between the existing pipe and the liner pipe.

The pipe or arch sections are then assembled outside the culvert and slid into place using winches, jacks or heavy equipment. Following placement of the liner, the ends are formed in preparation for grouting.

A sequence of photos showing replacement of an existing corrugated metal arch with a new corrugated metal arch in shown in Figure 14.16. The figure shows several steps, including (a) Assembly of plate arch outside existing culvert from interior of existing culvert, (b) Assembly of plate arch liner from outside existing culvert, (c) Plate arch liner in place prior to forming and grouting, (d) Exterior formwork for placing grout between liner and existing arches, (e) Formwork for grouting annular space between existing arch and liner from interior of liner, and (f) Interior close up showing how skewed end of corrugated arch was formed. For the installation shown in Figure 14.16, the grout was placed from the roadway surface by excavating to the top of the existing corrugated arch and cutting holes for grout placement (see Figure 14.17). Specifically in Figure 14.17, the liner arch and hole cut in existing arch visible in bottom of hole is shown (left) and placement of grout is shown (right). The completed lined arch is shown in Figure 14.18.

Figure 14.16 Sequence of Culvert Lining Replacement in Corrugated Metal Arch (Courtesy of Washington County NY DPW)

Figure 14.17 Grout Placement Hole Excavated in Road Surface (Courtesy of Washington County NY DPW)

Figure 14.18 Completed Lined Arch (Courtesy of Washington County NY DPW)

Figure 14.19 and Figure 14.20 show slip lining installations using smooth interior metal pipe. In Figure 14.19, the external view is shown (left) and internal view showing grout ports is shown (right). While the pipe in Figure 14.19 does have ribs, they are flat enough that the pipe is considered smooth for hydraulic purposed. In the installation of the metal pipe shown in Figure 14.20, grouting occurred from the interior using the grout ports shown in the photo. Following curing of the grout, the forms were removed and the stream was restored.

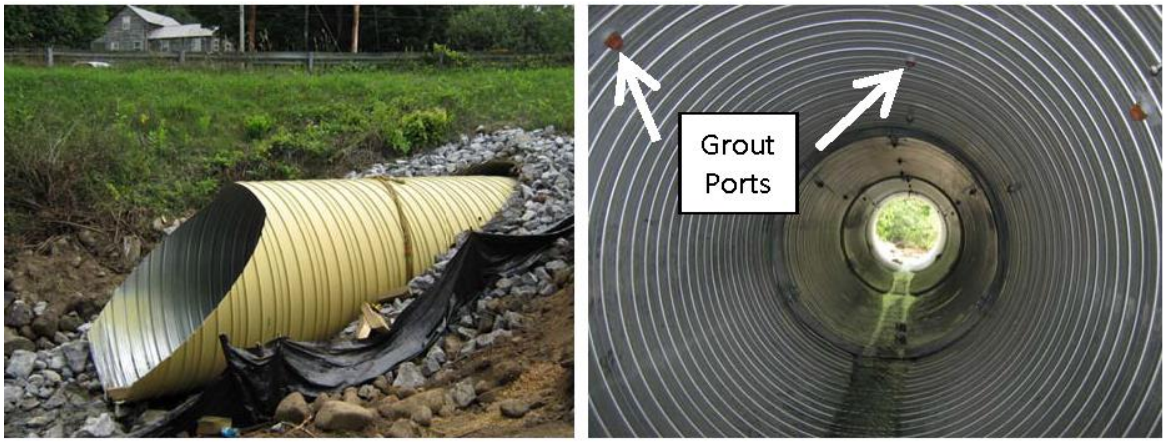

Figure 14.19 Smooth Interior Metal Pipe Slip Liner
(Courtesy of Saratoga County NY DPW)

Figure 14.20. Smooth Wall Steel Culvert Liners (Courtesy of Virginia DOT)

14.6.2.2 Shotcrete/Spin cast Concrete Liner

A deteriorated culvert may be relined with pneumatically projected concrete (Shotcrete) or spin cast concrete. Both methods provide a new concrete liner to the culvert which can be structural if designed correctly.

Pneumatically projected concrete is most often used in culverts that are large enough for workers to enter. It consists of a dry concrete mix that is pneumatically applied, typically 2 inches to 4 inches thick on the interior surfaces of the culvert. An example of shotcrete placement for culvert lining is shown in Figure 5.42. If necessary, reinforcing steel or sheer studs can be placed. Alternatively, steel or polypropylene fibers can be included in the concrete for added strength.

Spin cast concrete is typically used for smaller diameter culverts that are not feasible for workers to enter. The spin casting process uses a centrifugal device to cast a cement/mortar mix against the walls of the culvert to form a new concrete liner inside the existing culvert.

14.6.2.3 Cured In Place Pipe (CIPP)

Cured In Place Pipe (CIPP) may be used for reconstruction of culverts up to 108 inches diameter. The method installs a resin-impregnated, flexible tube which is inverted into the existing culvert by use of a hydrostatic head or air pressure. The resin is cured by circulating hot

water, steam, or hot air within the tube. When cured, the finished pipe will be continuous and tight-fitting. This technique requires taking the culvert out of service for the duration of installation and curing.

An example of the steps for installation of an inversion liner is shown in Figure 14.21, showing (a) Incisions are cut in sides of liner to facilitate inversion, (b) Sides of liner fastened to installation scaffolding, while first portion of liner is inverted, (c) Water is added to inverted liner to advance it toward installation manhole (d) Inverted liner being lined up and advanced into the installation manhole.

A completed installation of cured in place pipe is shown in Figure 14.22.

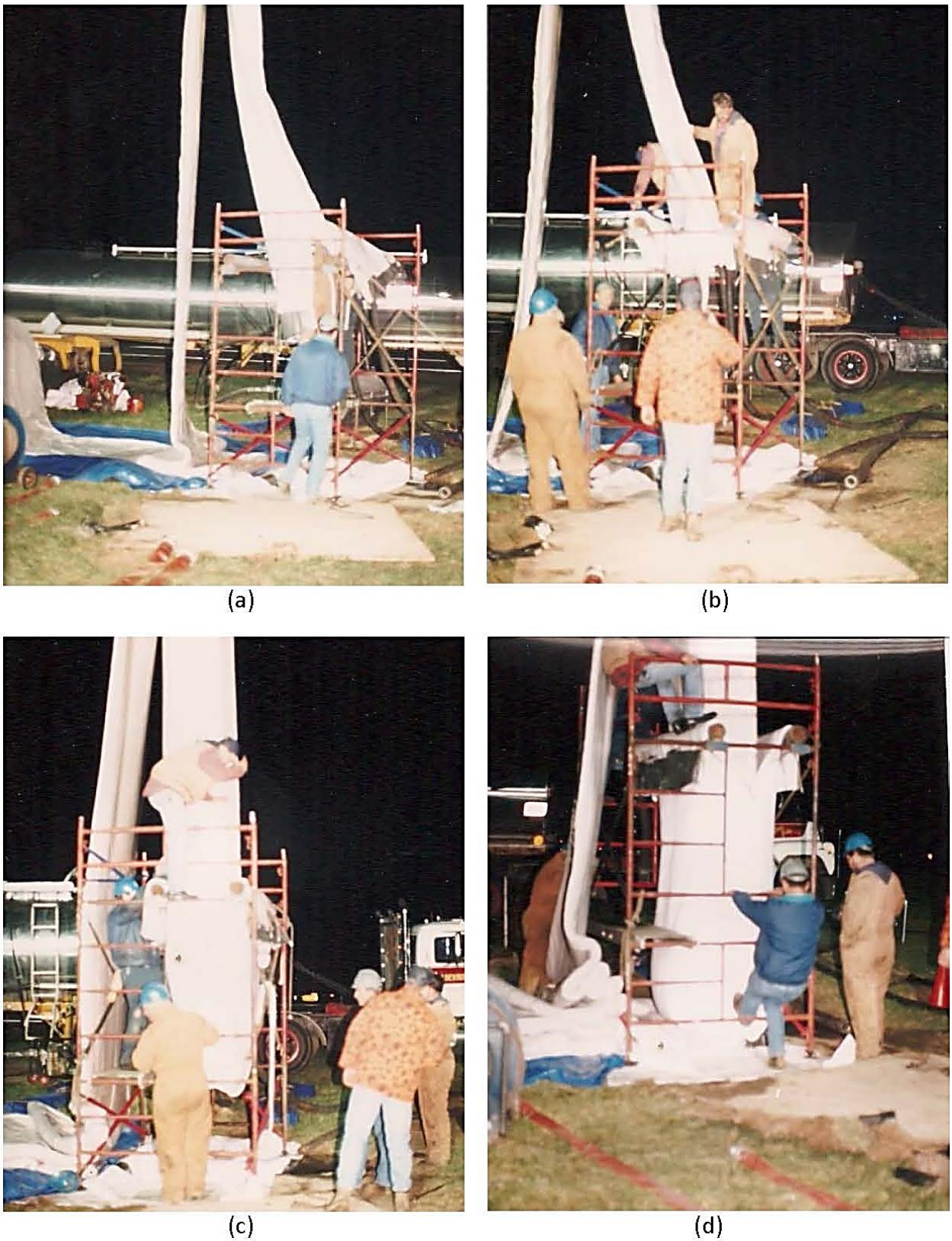

Figure 14.21 Inversion Liner Installation

Figure 14.22 A Completed Installation of Cured in Place Pipe (Courtesy of Insituform USA)

For information regarding spall and delamination repair, please see Chapter 5.

14.6.3 Joint Repair

Methods for joint repair include:

- Internal bands
- Internal joint seals
- Flexible chemical grout injection
- Concrete collars

These methods are described below.

Internal bands can be used for metal pipes where water tightness is not critical, but a soil tight mechanical connection is desired. Internal expanding metal bands are placed over the joint, and the bolts/threaded rod assembly is tightened forcing the bands against the surface of the pipe.

Suggested Procedure

Installation of Internal Expanding Bands

1. The area to be repaired is first cleaned.
2. The pipe is marked on either side of the joint or damaged area to ensure proper alignment.
3. The expanding band is placed over the center of the joint or damaged area.
4. The internal expanding metal bands are tightened, pressing the seal against the pipe.

Internal joint seals are comprised of an Ethylene Propylene Diene Monomer (EPDM) rubber gasket and metal band which expands to conform to the inner wall of the pipe. They can be used for both High Density PolyEthylene (HDPE) and metal pipes.

Suggested Procedure

Installation of Internal Joint Seals

1. The area to be repaired is first cleaned.
2. The pipe is marked on either side of the joint or damaged area to ensure proper alignment.
3. The seal is placed over the center of the joint or damaged area.
4. The expanding band is placed over the center of the seal.
5. The internal expanding metal bands are tightened, pressing the seal against the pipe.

Injection of flexible chemical grout creates a collar around leaking pipe and joints. Grouting chemicals are forced into the joint and the surrounding soil where they activate, sealing the joint. To inject the chemical grout:

Suggested Procedure

Injection of Chemical Grout

1. The culvert is first cleaned.
2. The equipment containing the grout is inserted into the pipe and positioned over the joint.
3. The grout is forced through the joint from the inside of the pipe and then gels when it comes in contact with water to form a waterproof collar.

Concrete collar installation involves building a form around the area to be repaired and encasing it in concrete.

Suggested Procedure

Construction of a Reinforced Concrete Collar

1. Excavate bedding from under and around the existing pipe to a width at least 12 inches from either side of the joint.
2. Encase the damaged joint with a concrete collar a minimum of 24 inches wide and at least 6 inches thick around the culvert.
3. Carefully replace and compact bedding.
4. Then backfill to provide proper support for pipe and collar.

An example of a concrete installation procedure is shown in Figure 14.23.

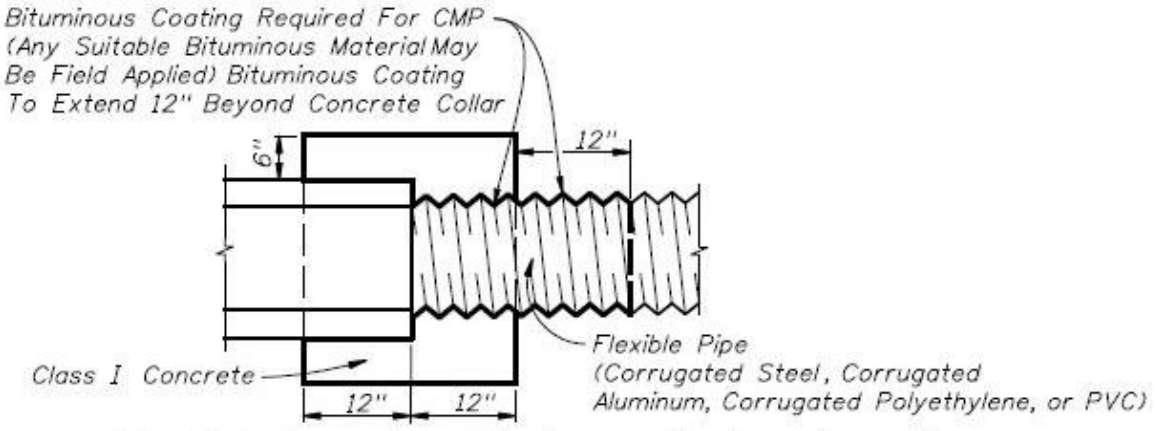

Figure 14.23 Concrete Collar Detail (Courtesy of Florida DOT)

14.6.4 Rehabilitating/Installing Toe Walls

In some cases, installation of toe walls can serve to create a permanent repair in areas where there is a tendency for scour holes to develop at either the inlet or outlet of a culvert. If sized correctly, toe walls prevent scour from undermining the culvert, but toe walls will not prevent scour holes from forming. The toe wall should extend to at least 6 inches below the depth of expected scour to protect the culvert from undermining. After the stream flow is diverted around the work area, the toe wall is formed and poured. The toe wall should be adequately anchored to the culvert with rebar, bolts, shear studs or other appropriate means. On completion of the wall, the scour hole is backfilled with either riprap, or native stream bed material to conform to the natural stream bed. An example of a toe wall and its associated detail are presented Figure 14.24 and Figure 14.25, respectively.

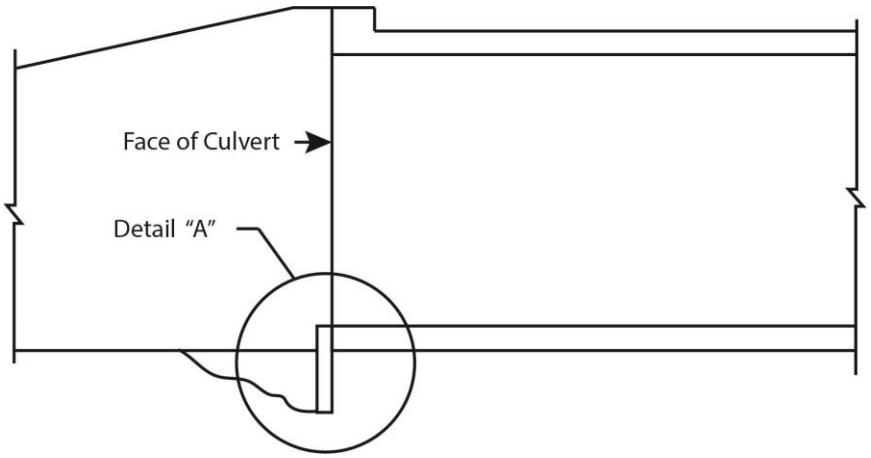

Figure 14.24 Toe Wall. Detail "A" Shown in Figure 14.25

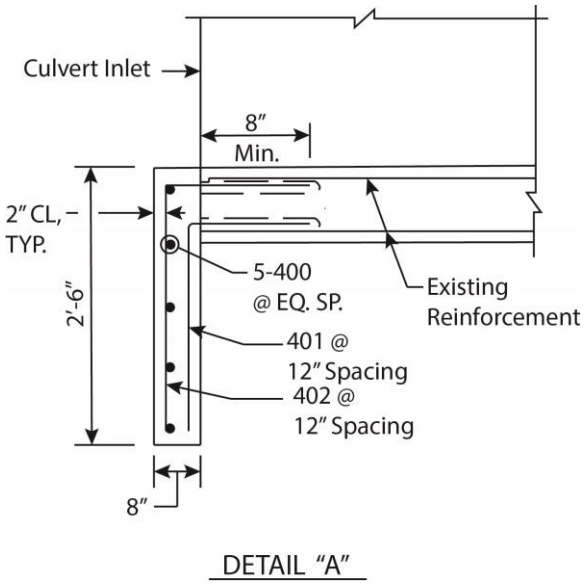

Figure 14.25 Toe Wall Detail at Concrete Box Culvert (Courtesy of GA DOT)

14.6.5 Repointing Masonry

Repointing of masonry is necessary when the mortar has deteriorated to the point that it is either missing or has loosened to the point where bonding and support are no longer provided.

Hand pointing is appropriate if the loss of mortar is less that the full depth of the masonry units, and the new mortar is primarily serving to protect and retain the original mortar. In the cases where the joint mortar is completely missing or deteriorated to the point where it is providing no support, pressure mortaring is a more appropriate technique. Pressure mortaring will provide full penetration of the joints to restore structural strength of the mortar.

Prior to repointing existing masonry, the existing mortar should be carefully evaluated, and a new mortar selected that will provide a similar strength to the existing mortar. A mixture of mortar strengths in the culvert could cause cracking of existing mortar joints, or cracks to propagate through masonry units where they had previously been in mortar joints.

Because of the age of many masonry structures, they may be eligible for the National Register of Historic Places, either alone or as part of a grouping, district, or larger feature. If state or federal funds are being used for the work, the State Historic Preservation office should be consulted before initiating any work. For more information on repointing masonry, please see Chapter 16 on Masonry Arch Bridges.

Suggested Procedure

Repointing Masonry

1. The masonry should be carefully evaluated to determine the extent and types of repairs.
2. A grout placement plan that also identifies any reinforcement is developed.
3. Necessary water diversions/cofferdams are deployed to permit grouting and repointing to occur in a dry environment.
4. Joints to be repaired are cleaned with low pressure water.
5. Cleaned joints are blown clean with compressed air.
6. Any reinforcement required is installed (See Figure 14.26).
7. Masonry is pre-wet to Saturated Surface Dry (SSD) prior to grouting or hand pointing.
8. Pointing and grouting is performed maintaining the moisture and temperature necessary to provide adequate curing of mortar or grout.
9. Face of masonry units is cleaned prior to set up of mortar or grout.

Figure 14.26 Reinforcing Steel Installed Prior to Grouting Stone Masonry

14.6.6 Footing Repair of Bottomless Culverts

In bottomless culverts, repair of the footing is similar to repair of concrete in piers or abutments (see Chapters 5 and 13). However, culvert footing repair often entails tight spaces

that make use of heavy equipment difficult or impossible, and diversion of stream flows where conventional cofferdams are not feasible.

The repair to a bottomless culvert footing requires assessing the extent of the repair by determining the limits of sound concrete and determining the design scour elevation. Aside from simple repairs to above water cracks and spalls, more extensive concrete repairs should extend below design scour depth if feasible.

To excavate, form, and pour in dry conditions the stream will need to be diverted. In Figure 14.27, the owner is using twin HDPE culverts to divert flow during a low flow period. Other diversion methods could include sandbagging, bypass pumping, cofferdams or Aqua dams.

Figure 14.27 Preparation for Footing Repair on Corrugated Plate Arch

Once the water has been diverted, the existing footing surface should be removed to sound concrete, and the soil at the toe of the footing should be excavated to the design repair depth.

As part of the forming process, provision should be made for adequately connecting the repair concrete to the existing footing. Typically, reinforcing bar is drilled and grouted into the existing concrete. Some agencies will treat the face of the existing concrete with epoxy or another bonding agent to strengthen the bond between the new concrete and the old concrete.

The concrete is placed and cured, forms removed, and the stream restored to its original course. The completed repaired footing is shown in Figure 14.28.

Figure 14.28 Repaired Footing for Corrugated Plate Arch

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Chapter 15 - Channel and Waterway

15.1 Identifying Scour and Erosion – Waterway Mechanics

Scour that results in the undermining of foundation units is one of the most common causes of bridge failure. Streams are dynamic; they change position, shape and other characteristics with variation in flow and the passage of time. When a channel is modified near a bridge, or a new bridge/culvert is installed, this local change will often lead to a change in the channel characteristics both up and downstream of the bridge. Similarly, channel modifications up or downstream of a bridge can affect the channel characteristics at the bridge.

The tendency of a stream channel to scour and migrate is dependent on many factors, including:

- Stream Size
- Stream Velocity
- Flow Habit (Ephemeral, Flashy, Perennial)
- Bed and Bank Material
- Valley, Floodplain & Natural Levee configuration
- Channel Pattern (Meandering, Straight, Braided, Anabranched)

The amount of material that is transported, eroded, or deposited in a stream channel is a function of sediment supply and channel transport capacity. This concept is described in Figure 15.1 with a definition sketch of sediment continuity applied to a given channel reach over a given time period. Sediment supply is provided from the tributary watershed and erosion occurring in the upstream channel. Transport capacity is a function of the sediment size, stream velocity and geometric properties of the channel. When the transport capacity equals sediment supply, a state of equilibrium exists. When this equilibrium is disturbed, erosion, scour, or deposition will occur.

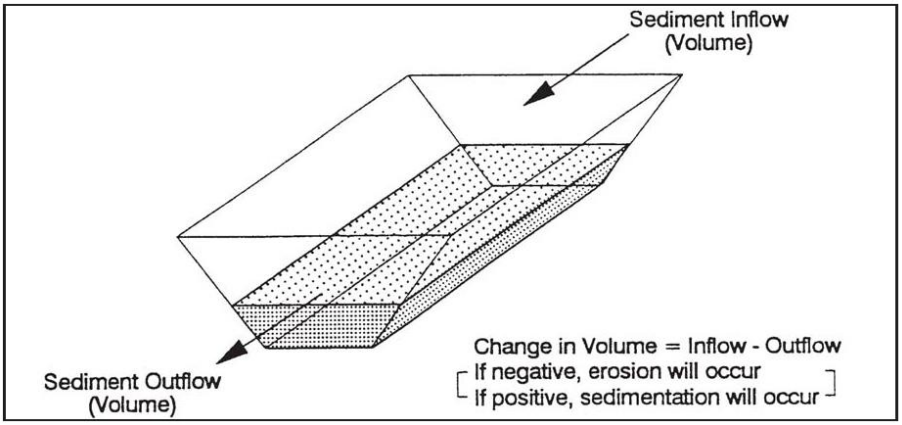


Figure 15.1 Definition Sketch of Sediment Continuity Concept

Migration occurs along a channel reach as bank materials are eroded at the outside of bends and deposited along the inside of bends, as shown in Figure 15.2. The figure shows modes of meander loop development, including (a) Extension, (b) Translation, (c) Rotation, (d)

Conversion to a Compound Loop, (e) Neck Cutoff by Closure, (f) Diagonal Cutoff by Chute, and (g) Neck Cutoff by Chute. Over time, migration can change the flow direction and alignment, thus affecting the efficiency of a bridge opening. Active bank erosion can be recognized by falling vegetation along the bank line, cracks along the bank surface, slump blocks (i.e., loosely consolidated materials or rock layers moving a short distance down a slope as a relatively coherent mass), increased turbidity, bent tree trunks and fresh vertical faces, as shown in Figure 15.3.

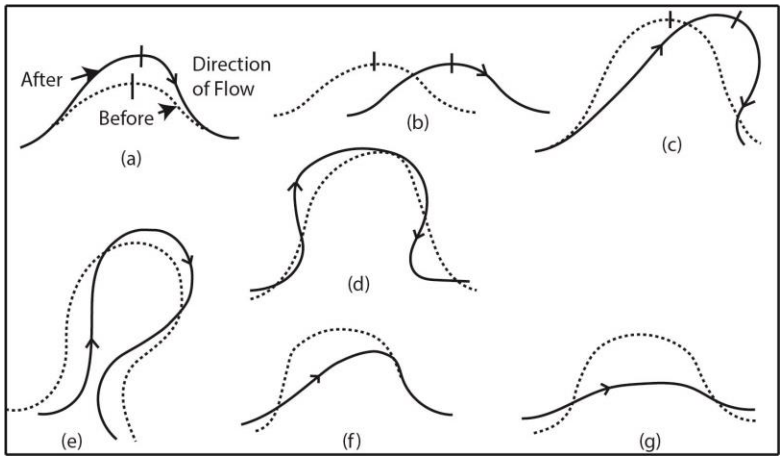


Figure 15.2 Modes of Meander Loop Development

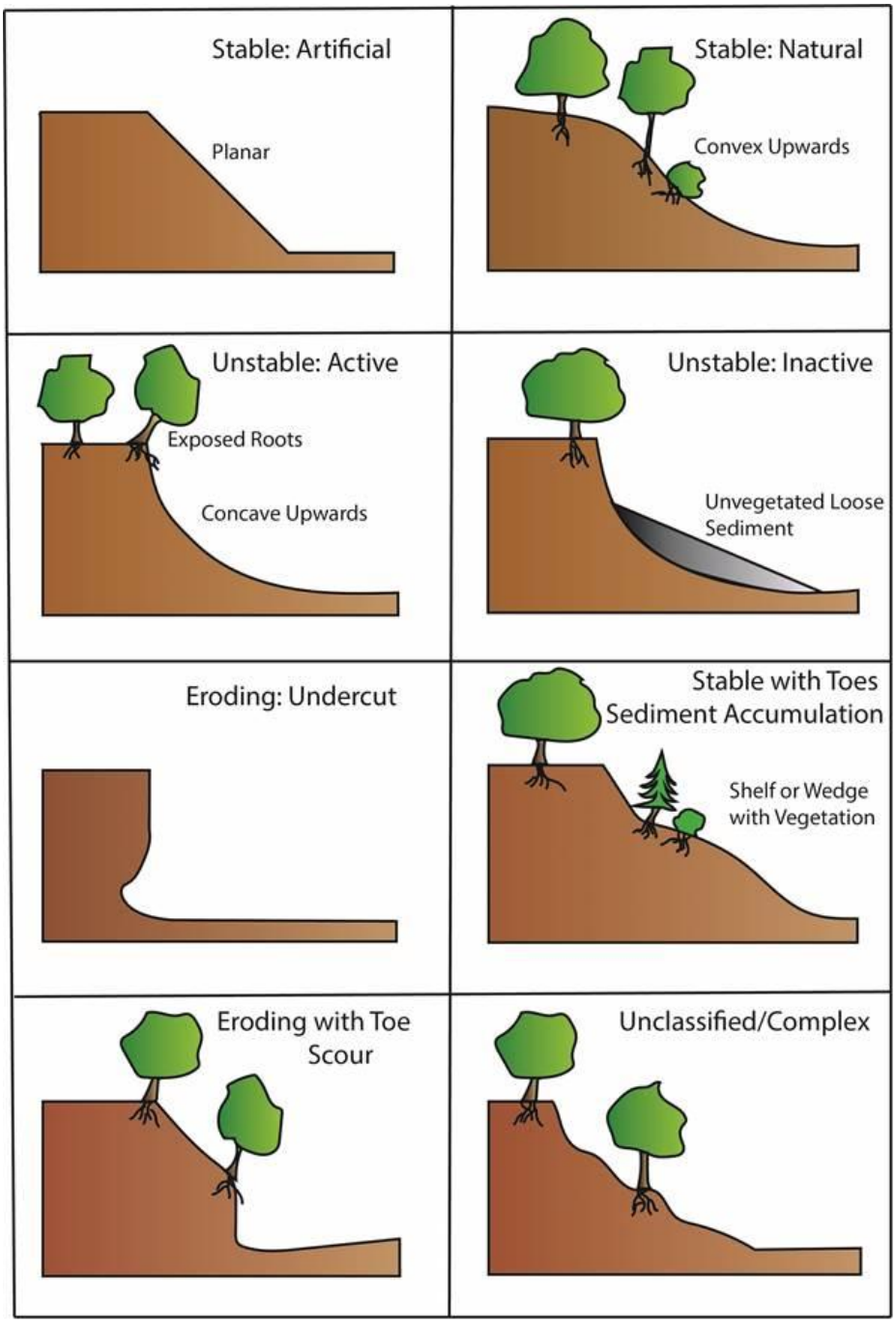


Figure 15.3 Classification and Morphology of Typical Bank Profiles

Aggradation and degradation are the vertical raising and lowering, respectively, of the streambed over relatively long distances and timeframes. The degree of aggradation and degradation are related to the sediment transport behavior of the waterway. Degradation can lead to the exposure and undermining of bridge foundations and increased susceptibility to local scour. Aggradation will reduce the hydraulic opening available at the bridge. Bed elevation changes can be recognized by taking periodic fascia sounding measurements from a fixed reference point and comparing the readings over time, i.e., channel profiles.

What To Look For

- Bank profiles that are not stable
- Evidence of scour around piers and abutments
- Evidence of debris creating problems around the bridge

Scour at bridges consists of three additive components :

1. Long term aggradation and degradation of the stream channel.
2. Contraction scour due to constriction or the location of the bridge, as shown in Figure 15.4. The figure shows an example of flow structure including macro-turbulence generated by floodplain/main channel flow interaction, flow separation around abutment, and wake region on the floodplain of a compound channel.
3. Local scour, as shown in Figure 15.5 for a cylindrical pier.

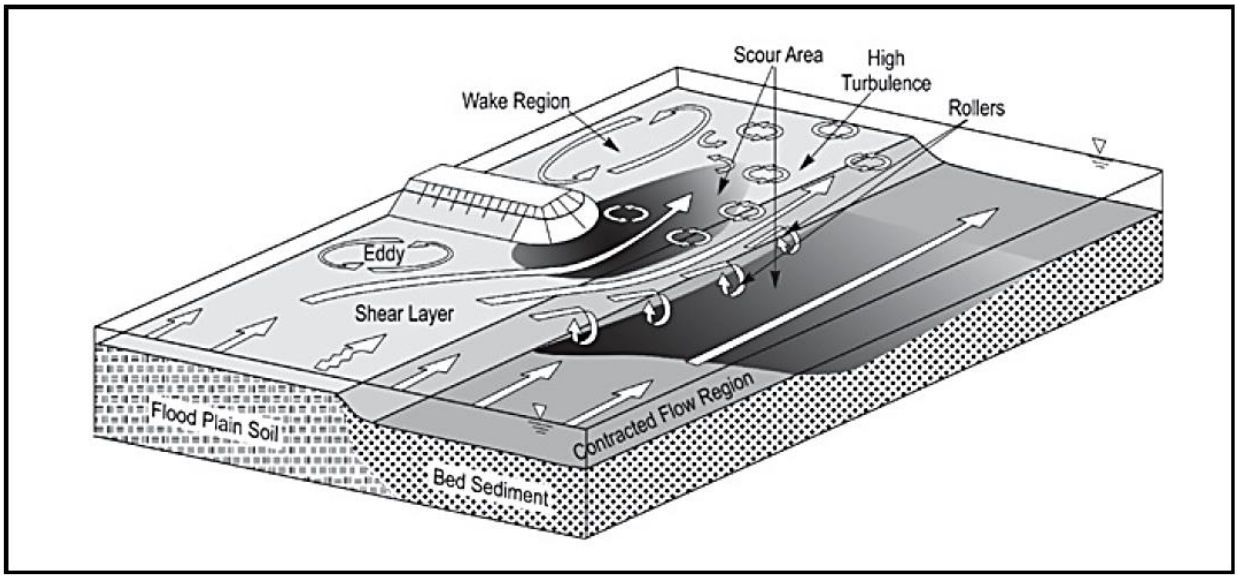


Figure 15.4 Construction Scour Example

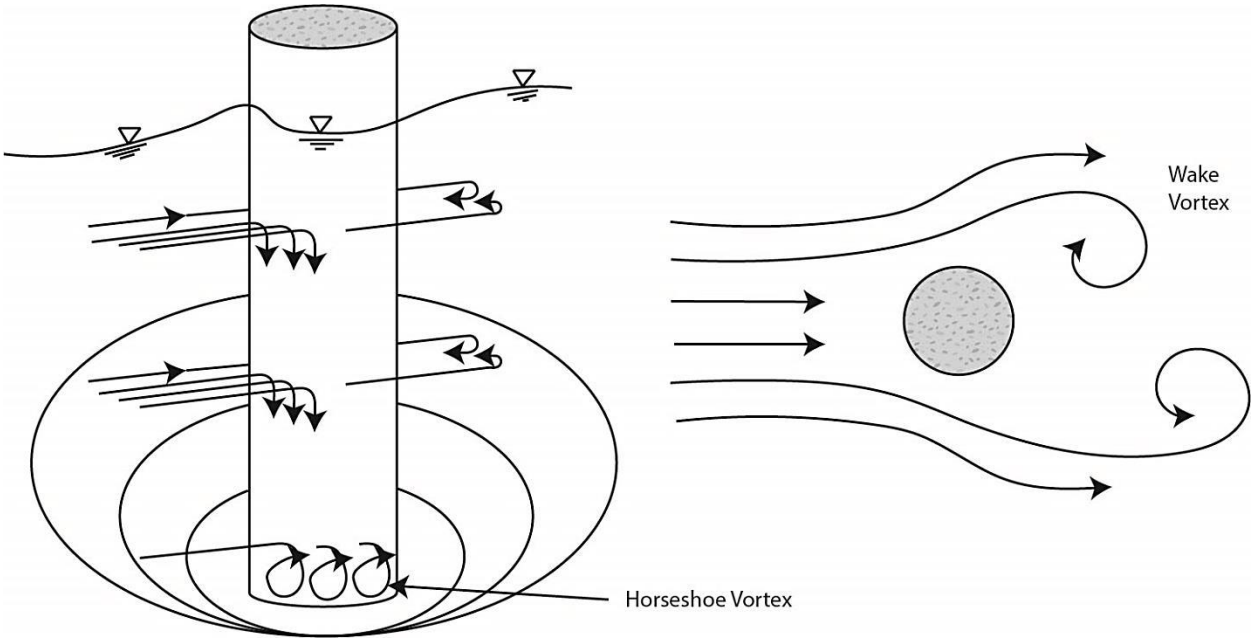


Figure 15.5 Schematic Representation of Scour at a Cylindrical Pier

The rate of scour depends on the erosive forces exerted on the channel boundary and the resistance of the material to erosion. Detailed information on waterway mechanics can be found in the FHWA Publication No. FHWA-NHI-12-004 *Stream Stability at Highway Structures* which can be downloaded from the FHWA web site. Additional resources include *Hydraulic Engineering Circular (HEC) 18* and *HEC 20*, the details of which are provided in the Reference List at the end of this chapter.

15.2 Preventive and Basic Maintenance for Channels and Waterway

15.2.1 Debris Removal

Waterborne debris (drift), composed primarily of tree trunks and large limbs, often accumulates at bridge piers, abutments and under-deck elements. High water events can increase the amount of debris in the waterway and accelerate debris accumulation. Accumulated debris restricts and redirects water flows through the bridge opening leading to flooding, damaging loads, and foundation scour, as shown by the example in Figure 15.6. Note that scour was defined and discussed in Section 4.8.3 Scour.

Consideration should be given to schedule debris removal to occur prior to NBIS regulated underwater bridge inspection operations. Accumulated debris is a diver safety hazard and will limit diver access to areas with high scour susceptibility.

Figure 15.6 Increased Scour at Bridge Piers as a Result of Debris

An effective debris removal program can improve the efficiency of the bridge opening and reduce scour potential during the next high water event.

Debris removal operations may be environmentally sensitive, as shown in Figure 15.7 and Figure 15.8. Coordination with local environmental regulatory agencies may be required for non-emergency work. Best management practices for debris removal include:

- Only cut material when necessary. Turn the debris to allow it to flow through the structure if downstream structures will not be affected; note this practice may not be environmentally acceptable for all bridge owners.
- Minimize machinery disturbance to the stream bank and bed materials. Where possible, use specialized equipment such as a crane, operated from the bridge deck.
- Perform any required work in flowing channels during the work window permitted for the specific stream body.
- Repair and restore bank areas which may be impacted by drift removal machinery.

Recommendation

Accumulated debris can create vortices and accelerate local scour. After debris removal, consider performing a scour inspection to determine if foundation exposure or undermining repairs are also required.

Figure 15.7 Debris Removal Operation
(Courtesy of the City of Rochester, New York - Department of Environmental Services)

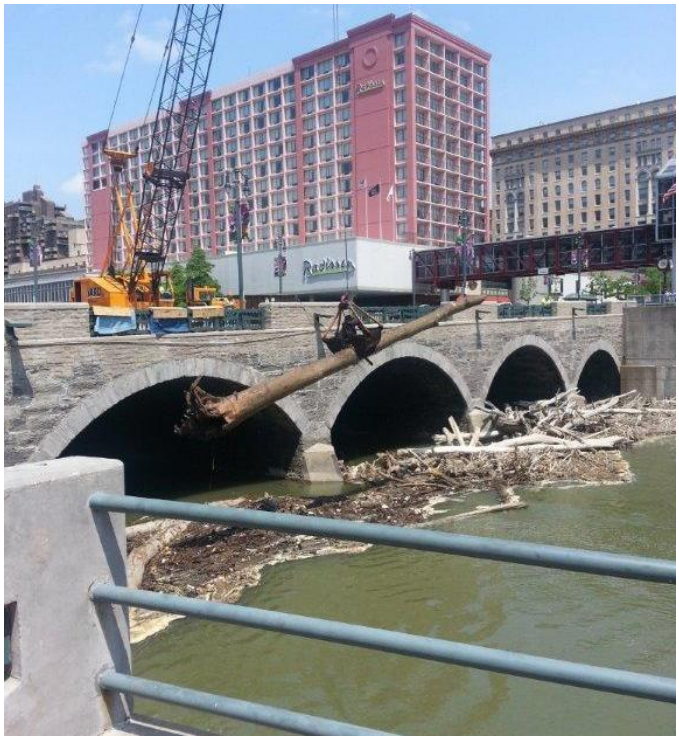


Figure 15.8 Debris Removal Operation
(Courtesy of the City of Rochester, New York - Department of Environmental Services)

If a structure has a tendency to accumulate debris and is configured such that debris removal is difficult, debris deflectors can be constructed near the structure. Alternatively, containment gates can be constructed at a convenient location upstream, as shown by the examples in Figure 15.9 and Figure 15.10. More specifically, Figure 15.9 shows a post and rail debris rack, in place for 35 years for light to medium floating debris, installed 100 feet upstream of the culvert. Figure 15.10 shows timber debris fins with sloping leading edge.

When to Call the Engineer

Containment gates can significantly affect upstream flood elevations. A hydraulic analysis should be performed prior to installing this type of countermeasure to assess the impact on upstream properties and structures.

Figure 15.9 Post and Rail Debris Rack Installed 100 Feet Upstream of Culvert

Figure 15.10 Timber Debris Fins with Sloping Leading Edge

15.2.2 Vegetation Removal

The condition of the vegetation and surroundings should be evaluated to determine the appropriate method and degree of vegetation removal. Depending on the type of channel and the conditions, the vegetation removal may be as simple as mowing the areas to keep them clear, without increasing erosion potential.

In areas where fast growing vegetation is a concern, or obstruction of the waterway with vegetation has been a recurring problem, mechanical removal (excavation of the roots) or use of herbicides may be appropriate. Use of herbicides will be governed by local environmental regulations.

15.3 Repair and Rehabilitation of Channel and Waterway

Knowledge of the mechanics of stream migration and erosion can be used to predict future conditions and implement repairs for observed problems. Effective countermeasures can be

employed to control stream migration and to protect foundations from local scour. Extensive information on the selection and design of countermeasures can be found in FHWA Publication No. FHWA-NHI-09-112 (Bridge Scour and Stream Instability Countermeasures: Experience, Selection and Design Guidance) which can be downloaded from the FHWA web site.

Maintenance countermeasures typically consist of river training and armoring techniques which are designed either to modify the flow or resist erosive forces.

River Training Structures are those which modify the flow by altering stream hydraulics to mitigate undesirable erosional and or depositional conditions at a particular location or in a river reach. This type of structure is typically classified by its orientation to flow:

- **Transverse Structures** are countermeasures which project into the flow field at an angle or perpendicular to the flow (bendway weirs, groins, check dams)
- **Longitudinal Structures** are countermeasures which are oriented parallel to the flow or along a bank-line (bulkheads, toe-dikes, levees)
- **Areal Structures** are other countermeasure treatments such as channelization, flow relief, and sediment detention

Armoring Countermeasures resist the erosive forces caused by a hydraulic condition. This type of countermeasure is typically classified by the armoring material and location. These systems can be either rigid or flexible. Rigid systems are typically impermeable but are subject to undermining failure. Flexible systems can conform to changing conditions but are subject to removal and displacement failure. Types of armoring include:

- **Revetments** are designed to protect the channel bank.
- **Bed Armoring** is designed to protect the river bed.
- **Local Scour Armoring** is designed specifically to protect individual substructure elements from local scour.

Other countermeasure strategies include structural (e.g., foundation strengthening, pier geometry modification), biotechnical (e.g., vegetated geofabrics, woody mats, root wads) and monitoring. A convenient reference guide is provided in *Appendix B: References* of this manual on a wide range of countermeasures applicable to scour and stream stability problems. The table can also be used to identify and prioritize possible alternatives.

Contraction Scour Countermeasures–The most typical countermeasures used for preventing contraction scour are revetments placed on the embankment fill slopes and spur dike/guide bank construction at the upstream side of the abutments.

Local Scour Countermeasures–Countermeasures can include modifications to move the scour away from the foundations, pier nose streamlining as well as armoring. The practice of heaping stones around a pier is not recommended because experience has shown that continual replacement is usually required. An engineered armoring system (detailing size, location, shape, and filtering) will result in a more robust protection.

Aggradation / Degradation Countermeasures–Check dams and detention structures can effectively be used to control bank slipping and caving and sediment deposits near bridges. In

lieu of permanent structures, continuing maintenance (dredging / clearing of deposited material) has been used effectively to remove deposited sediment and maintain the hydraulic openings at bridges, as shown in Figure 15.11.

When to Call the Engineer

An engineer should be contacted when designing any of these treatments.

(a) (b)

Figure 15.11 Deposited Sediment Removal to Maintain Hydraulic Opening (a) Before, (b) After (Courtesy of PA Department of Transportation)

Many countermeasure strategies involve work in streams which may be regulated by local environmental agencies. Consultation with these agencies is advised for non-emergency actions. Discharge of fill, placed within waters of the United States, is regulated through the United States Army Corps of Engineers (USACE) through Section 404 of the Federal Clean Water Act. Nationwide Permits are issued typically every four years to authorize common work items which have been determined to have minimal adverse impacts. The scope of proposed work items should be checked against the most recent issuance of Nationwide Permit No's 3 (Maintenance of Currently Serviceable Structures) and 13 (Bank Stabilization). If a bridge foundation has been determined to be scour critical as part of the bridge owner's scour evaluation program, the USACE will give priority to the bridge owner's request for authorization of the installation of scour countermeasures. Advanced notice of the proposed countermeasure design and construction schedule to the bridge owner and USACE is required.

In addition, the state environmental agency may also have jurisdiction over work done in and adjacent to waterways, so any channel or waterway work should also be reviewed for compliance with state environmental regulations. This not only applies to excavation and placement of fill (riprap), but may also apply to placement of cementitious materials in the waterway.

When to Call the Engineer

Call the Engineer to ensure countermeasures are adequately sized and located to perform correctly.

Countermeasures must be inspected periodically and after floods to check their performance. The design may need to be modified as a result of the inspection. The condition of the countermeasure should be documented with photographs to enable comparison between inspections. In most cases, countermeasures do not "cure" the scour or instability problem. Funding for continued maintenance of the countermeasures should be planned. Non-structural countermeasures are not considered permanent.

15.3.1 Placement of Riprap and Gabions

Riprap and gabion placements are flexible armoring techniques that can be used in various configurations to counter the erosive action of moving water. Riprap refers to a layer or facing of rock, placed on channel and structure boundaries to limit the effect of erosion. Gabions and mattresses refer to loose collections of stones encased in a wire framework (See Figure 15.12 through Figure 15.15).

- Revetments, bendway weirs and spur dikes to counter migration
- Bed armoring and check dams to counter degradation
- Revetments and guide banks to counter contraction scour
- Bed armoring to counter local scour

Riprap is a very common countermeasure due to its general availability, ease of installation, and relatively low cost. Riprap placements should be designed to resist particle erosion, substrate material erosion, and mass failure. Typical practices involve the use of filter fabric bedding, limiting slope angles, and trenching the toe below the anticipated contraction and degradation scour level.

The basis of protection provided by the riprap is the mass and interlocking of the individual rocks, therefore careful attention needs to be given to placement technique and mat thickness.

Riprap is not considered a permanent repair to scour and must be periodically monitored at a minimum on the two year inspection cycle.

The following figures are intended to present a basic insight into riprap and gabion uses. An extensive coverage of riprap and gabion countermeasures can be found in Chapter 5 of the FHWA publication, *Hydraulic Engineering Circular No. 23 – Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance*.

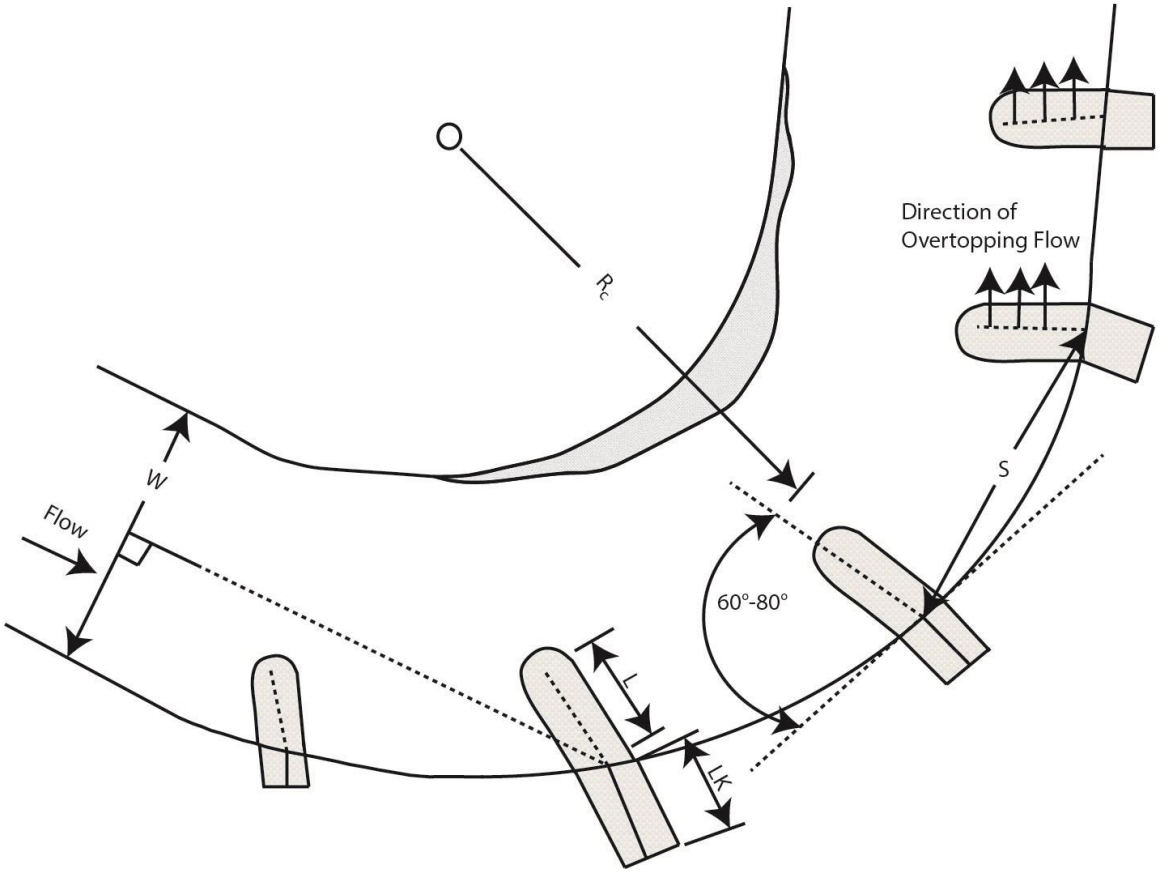


Figure 15.12 Bendway Weir Typical Plan View

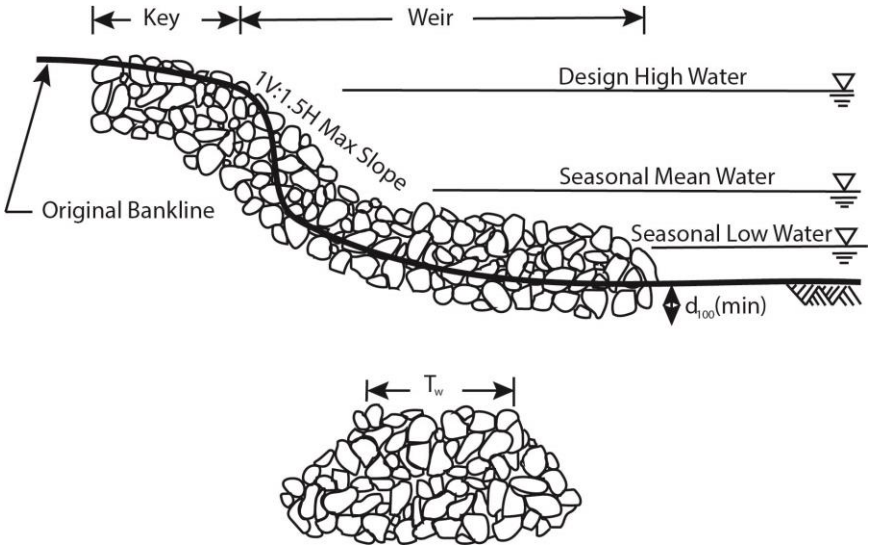
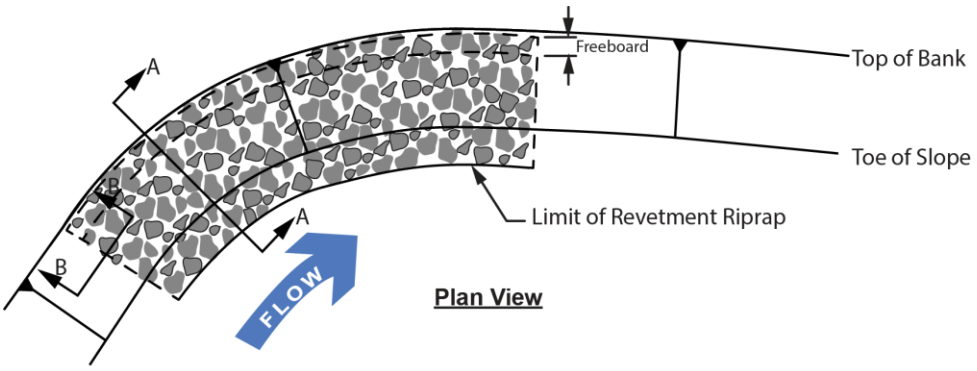
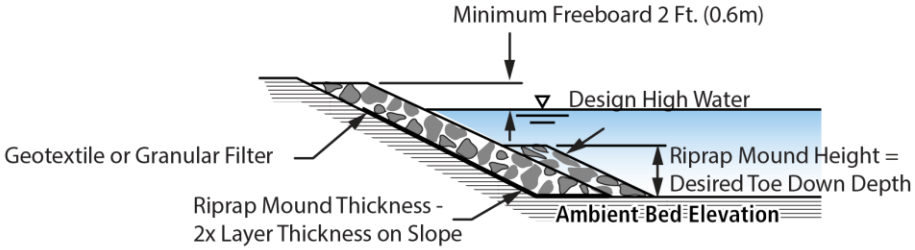


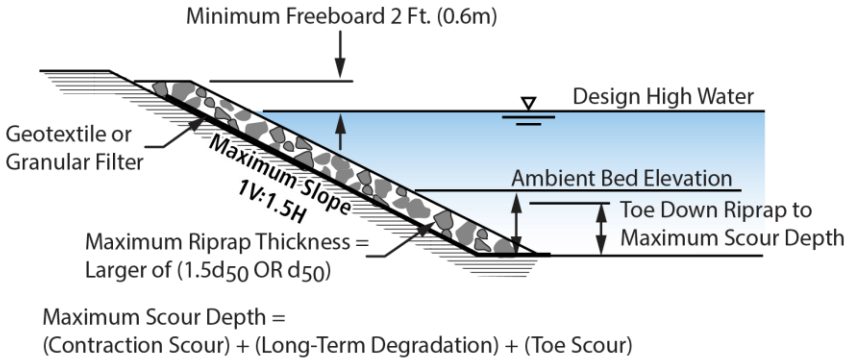
Figure 15.13 Bendway Weir Typical Cross Section



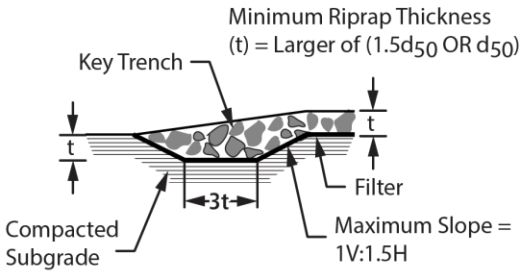
Plan View


Minimum Freeboard 2 Ft. (0.6m)
Design High Water
Geotextile or Granular Filter
Riprap Mound Height = Desired Toe Down Depth
Riprap Mound Thickness - 2x Layer Thickness on Slope
Ambient Bed Elevation

Section A-A
(Revetment Riprap Showing Mounted Toe Slope Termination)

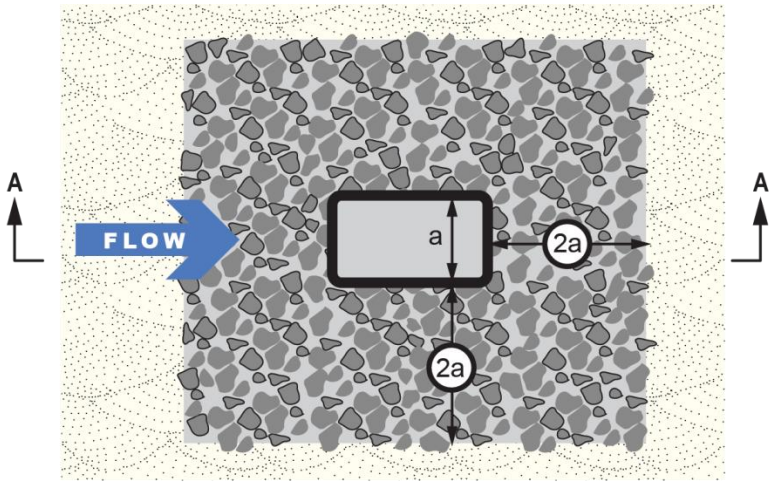

Minimum Freeboard 2 Ft. (0.6m)
Design High Water
Geotextile or Granular Filter
Maximum Slope 1V:1.5H
Ambient Bed Elevation
Toe Down Riprap to Maximum Scour Depth
Maximum Riprap Thickness = Larger of (1.5d50 OR d50)
Maximum Scour Depth = (Contraction Scour) + (Long-Term Degradation) + (Toe Scour)

Section A-A
(Revetment Riprap Showing Toe Down Slope Termination)



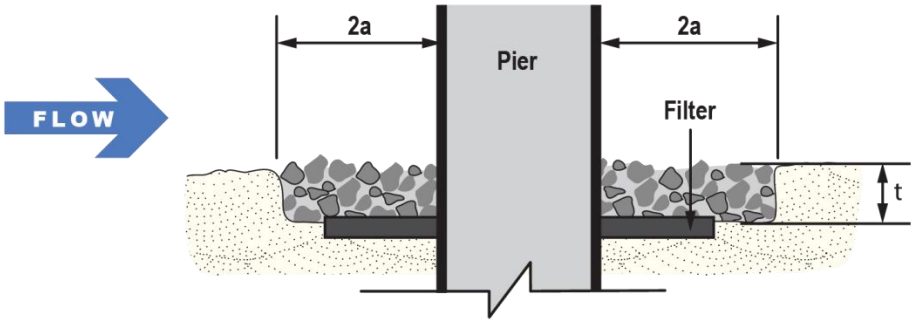
Section B-B

Figure 15.14 Riprap Revetment Details



Pier width = "a" (normal to flow)
Riprap placement = 2(a) from pier (minimum, all around)

Plan View



Minimum riprap thickness t=3d_{50}, depth of contraction scour and long-term degradation, or depth of bedform trough, whichever is greatest

Filter placement = 4/3(a) from pier (all around)

Section A-A

Figure 15.15 Riprap Layout Diagram for Pier Local Scour Protection

Because stone is produced and delivered in a range of sizes and shapes, the required size of stone is often stated in terms of minimum, mean, and maximum allowable size or weight. The shape of a stone can be generally described by designating three axes of measurement, as shown in Figure 15.16. Riprap stones should not be thin and platy, nor should they be long and needle-like. Therefore, specifying a maximum allowable ratio between the largest and smallest axis (i.e., shape factor) provides a suitable measure of particle shape. A maximum shape factor of 3.0 is recommended. Recommended tests for riprap rock quality are presented in Table 15.1.

Table 15.1 Recommended Tests for Riprap Quality

Test Designation	Property	Allowable Value	Frequency(1)	Comments
AASHTO TP 61	Percentage of Fracture	< 5 percent	1 per 20,000 tons	Percentage of pieces that have fewer than 50% fracture surfaces
AASHTO T 85	Specific Gravity (S_g) and Water Absorption	Average of 10 pieces: $S_g > 2.5$ Absorption < 1.0%	1 per year	If any individual piece exhibits an $S_g < 2.3$ or water absorption greater than 3.0%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
AASHTO T 103	Soundness by Freezing and Thawing	Maximum of 10 pieces after 25 cycles: <0.5%	1 per 2 years	Recommended only if water absorption is greater than 0.5% and the freeze-thaw severity index is greater than 15 per ASTM D 5312.
AASHTO T 104	Soundness by Use of Sodium Sulfate or Magnesium Sulfate	Average of 10 pieces: < 17.5%	1 per year	If any individual piece exhibits a value greater than 25%, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
AASHTO TP 58	Durability Index Using the Micro-Deval Apparatus	Value: Application > 90: Severe > 80: Moderate > 70: Mild	1 per year	Severity of application per Section 5.4, CEN (2002). Most riverine applications are considered mild or moderate.
ASTM D 3967	Splitting Tensile Strength of Intact Rock Core Specimens	Average of 10 pieces: >6 MPa	1 per year	If any individual piece exhibits a value less than 4 MPa, an additional 10 pieces shall be tested. If the second series of tests also exhibits pieces that do not pass, the riprap shall be rejected.
ASTM D 5873	Rock Hardness by Rebound Hammer	See Note (2)	1 per 20,000 tons	See Note (2)
Shape	Length to Thickness Ratio (A/C)	<10%, d_{50} < 24 inch <5%, d_{50} < 24 inch	1 per 20,000 tons	Percentage of pieces that exhibit A/C ratio greater than 3.0 using the Wolman Count method (Lagasse et al. 2006).
ASTM D 5519	Particle Size Analysis of Natural and Man-Made Riprap Materials		1 per year	See Note (3)
Gradation	Particle Size Distribution Curve		1 per 20,000	Determined by the Wolman Count method (Lagasse et al. 2006) where particle size "d" is based on the intermediate "B" axis.

(1) Testing frequency for acceptable of riprap from certified quarries, unless otherwise noted. Project-specific tests exceeding quarry certification requirements, either in performance value or frequency of testing, must be specified by the Engineer.

(2) Test results from D 5873 should be calibrated to D 3967 results before specifying quarry-specific minimum allowable values.

(3) Test results from D 5519 should be calibrated to Wolman Count (Lagasse et al. 2006) results before developing quarry-specific relationships between size and weight; otherwise assume W = 85 percent that of a cube of dimension "d" having a specific gravity S_g.

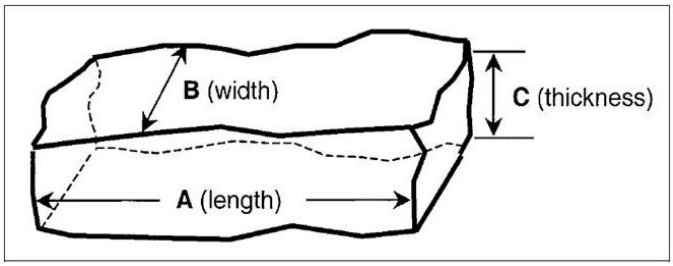


Figure 15.16 Riprap Shape Described by Three Axes

A general rule of thumb for riprap layer thickness is not less than the greater of the diameter of the largest allowable stone or 1.5 times the diameter of the median stone. Riprap placed underwater should be 50 percent thicker than the above noted criteria.

Advantages to gabions and mattresses are that the smaller stone (smaller than riprap) armoring can ease placement and is more resistant to removal by the waterway velocity. Mattresses are usually 1.0 foot or less in thickness, whereas gabions are thicker and nearly equidimensional. The use of these devices is most effective in transitory streams associated with an arid climate. Corrosion and abrasive failure of the cage wire is a concern for gabion construction in perennial streams. Galvanized and PVC coated wires are commonly used to extend the expected life span of gabions. Gabions are not recommended for use in gravel bed streams or in saline or acidic waterways. To improve anchorage, the cages can be anchored to steep slopes with railroad spikes. Examples of gabion mattress design details are presented in Figure 15.17 through Figure 15.20.

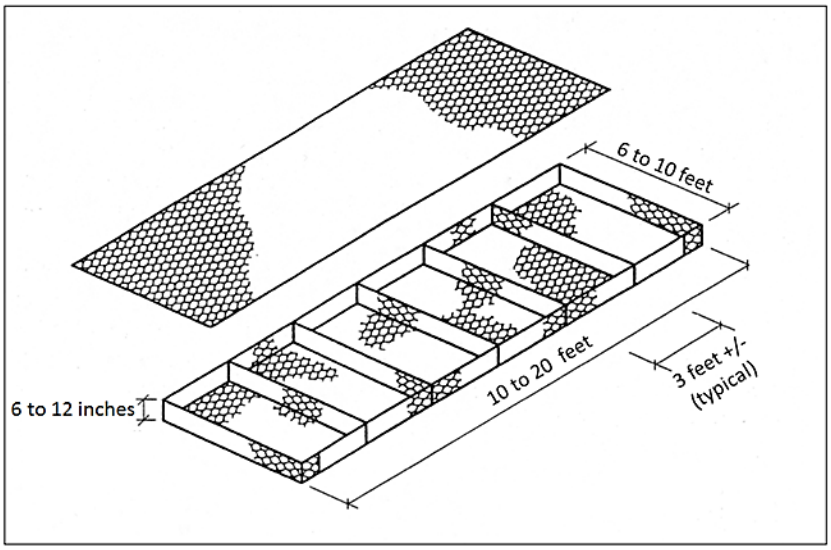


Figure 15.17 Gabion Mattress Showing Typical Dimensions

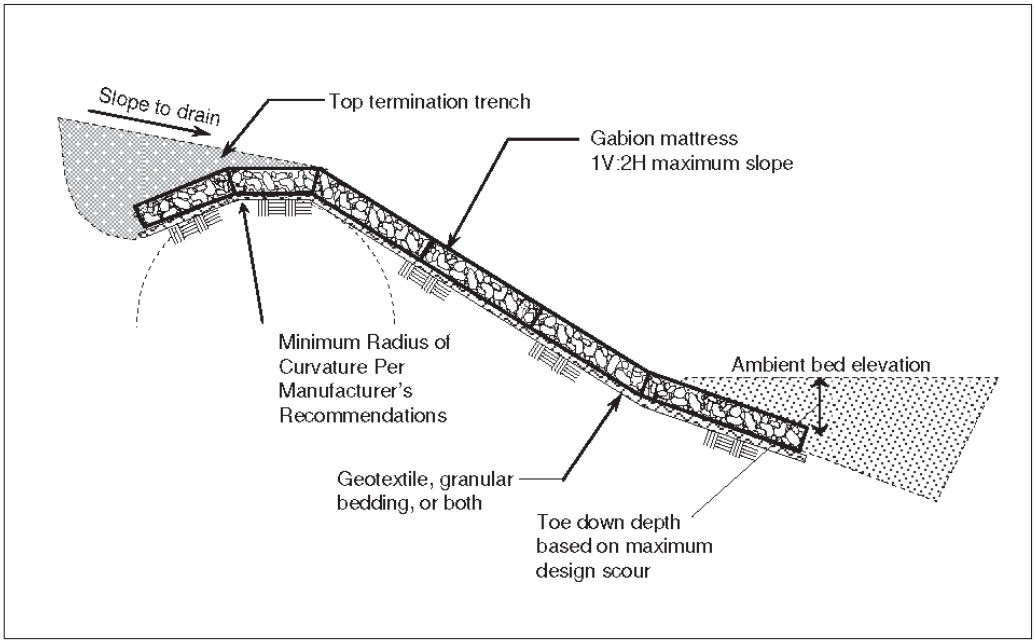


Figure 15.18 Suggested Installation Detail for Gabions Used as Bank Revetment

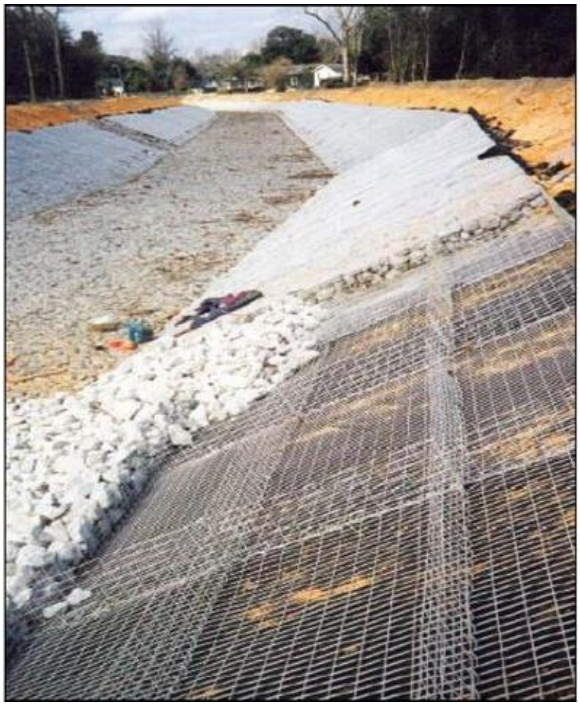


Figure 15.19 Field Installation of Gabion Mattresses on Channel Bed and Banks

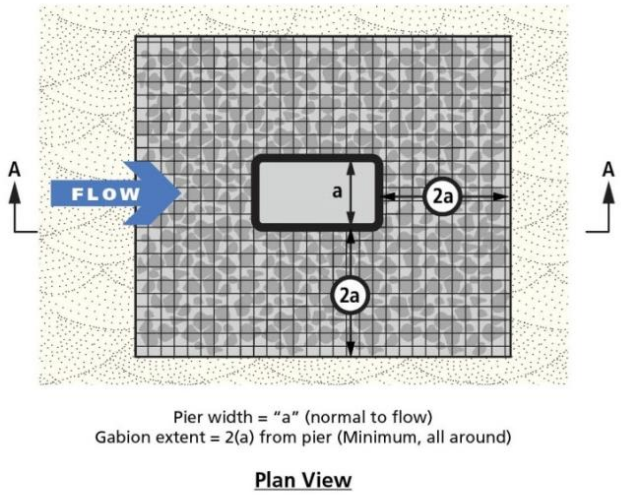


Figure 15.20 Gabion Mattress Layout Diagram for Pier Local Scour Counter Measures

Filters are essential to the successful long-term performance of porous armoring countermeasures such as riprap and gabions. The filter layer prevents excessive migration of the base soil particles through the voids in the armor layer, permits relief of water pressure under the armor, and distributes the weight of the armor to provide uniform settlement. There are two basic types of filters: granular and geotextile.

Subgrade soils should not be frozen and should be free of organic material. Divers should be used to verify underwater conditions are suitable for riprap placement. Filter placement should result in a continuous installation that maintains contact with the subgrade. Voids, gaps, and tears should be replaced or repaired.

Placing geotextiles under water is difficult. Some fabrics are buoyant and need to be anchored until the armor material is placed. Strong currents will create large forces on the fabric, causing the fabric to act like a sail, resulting in wavelike undulation. In deep water or strong currents, sand filled geotextile containers or proprietary mat systems can provide the filtering requirements. Geotextile should be placed so that upstream strips overlap downstream strips (1.5 feet dry / 3.0 feet underwater). Anchoring pins or weights can be used to keep the fabric in place.

Granular filters should be placed with front-end loaders with slopes limited to 1V:4H. Tremie tubes can be used to place granular filter material underwater at bridge piers.

Riprap may be placed underwater or in the dry from land or water based operations. Stones should be placed from the bottom working toward the top of the slope so that rolling and/or segregation does not occur. Riprap should be placed using methods that do not stretch, tear, puncture, or reposition the underlying fabric. Tracked and wheeled equipment should not be permitted to operate on lower lifts or finished application because they can destroy the interlocking integrity. Special purpose equipment, such as clamshells, orange-peel grapples, or hydraulic excavators, is available for installation. Sounding surveys, divers, sonar profiles, or remotely operated vehicles should be used to monitor and verify underwater placements.

Riprap and gabion mattresses can be grouted to control particle erosion and to create a smoother / more efficient hydraulic opening. Filters are not required for fully grouted riprap, however drainage of pore pressure must be provided. A significant disadvantage of complete grouting is that the grout converts a flexible revetment to a rigid cover, subject to toe undercutting, out-flanking, and the possibility of sudden catastrophic failure. Partially grouted systems have been used to maintain system flexibility while increasing stability (Figure 15.21). Guidance suggests that the grout should fill between 1/3 and 1/2 of the total void spaces. Additionally, a filter layer should be used with partially grouted rip-rap.

Figure 15.21 Close-Up View of Partially Grouted Riprap

Overhead restrictions should be considered when determining if riprap placement is a feasible alternative along abutment and pier faces obstructed by low superstructures. This is an example where gabions may be more practical than large riprap.

15.3.2 Tremie Concrete

Tremie concrete placement methods use a pipe or tube, through which concrete is placed below the water level. The lower end of the tube is kept immersed in fresh concrete, so that the rising concrete from the bottom displaces the water without washing out the cement content (see Figure 15.22). Tremie placements should be isolated from fast moving currents.

Figure 15.22 Tremie Placement (Courtesy of Underwater Engineering Services, Inc.)

The concrete mix should be designed to flow readily and the vertical fall distance should be minimized to prevent segregation. Concrete admixtures are available to improve the concrete mix performance for tremie applications. Examples are antiwashout admixtures and flow-improving admixtures.

Tremie placements can be effectively used to repair foundation undermining after a local scour event. An example is shown in Figure 15.23. A means should be provided to allow the trapped water to escape as the concrete surface rises.

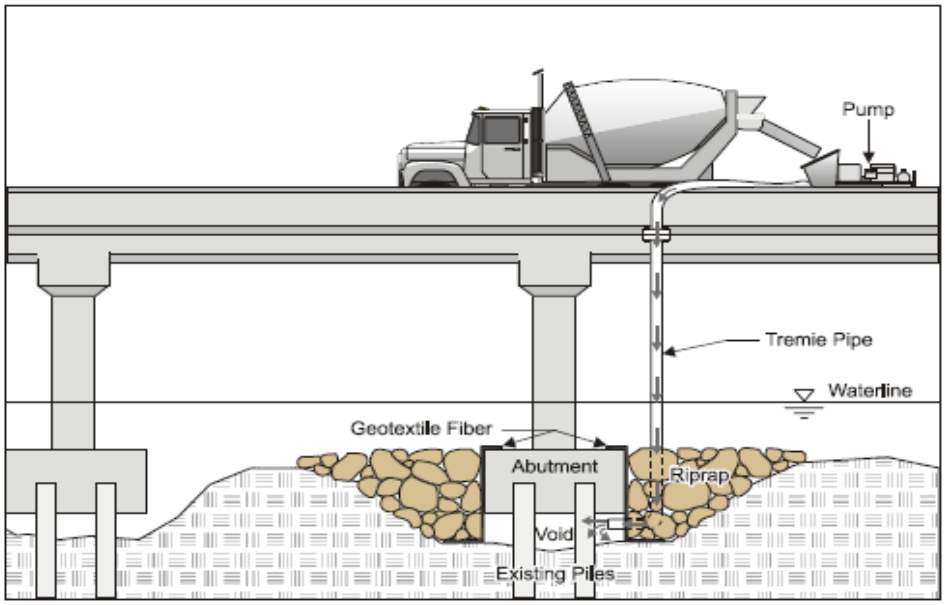


Figure 15.23 Tremie Concrete Placement used to Repair an Undermined Pile Cap (Courtesy Virginia DOT)

15.3.3 Grout Bag Placement

Concrete or grout filled bags and mattresses can be placed on dry bedding material or underwater by divers. The bags can be stacked and pinned together with rebar. The bags are

fabricated in many shapes, sizes, and configurations with many proprietary systems available for a wide range of applications.

Common applications for grout filled bags include riprap substitution (revetment armoring, groin construction, local scour armoring, etc.) and as facing formwork for foundation undermining repairs. Examples are shown in Figure 15.24 through Figure 15.28.

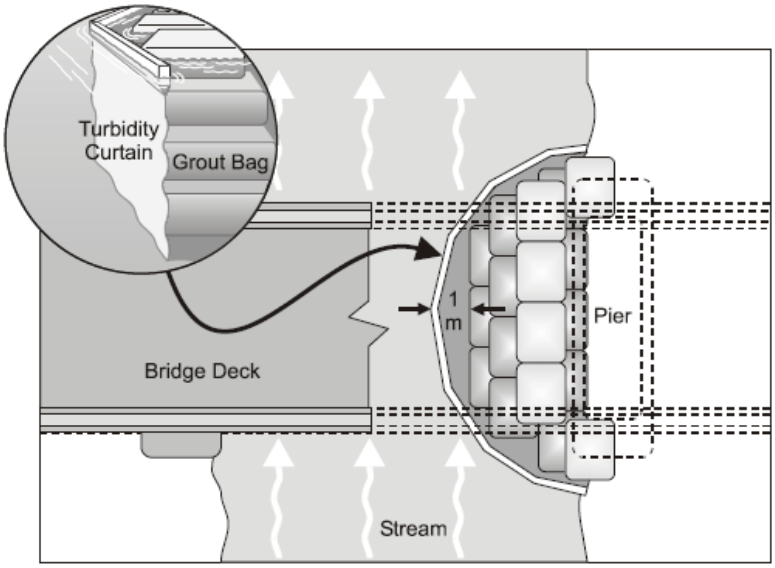


Figure 15.24 Grout Bags used for Local Scour Armoring (Riprap Substitution) (Courtesy Virginia DOT)

Figure 15.25 Grout Bags Installed around a Pier Nose (Courtesy Virginia DOT)

Figure 15.26 Grout-Filled Mat Used for Scour Protection at a Bridge Abutment

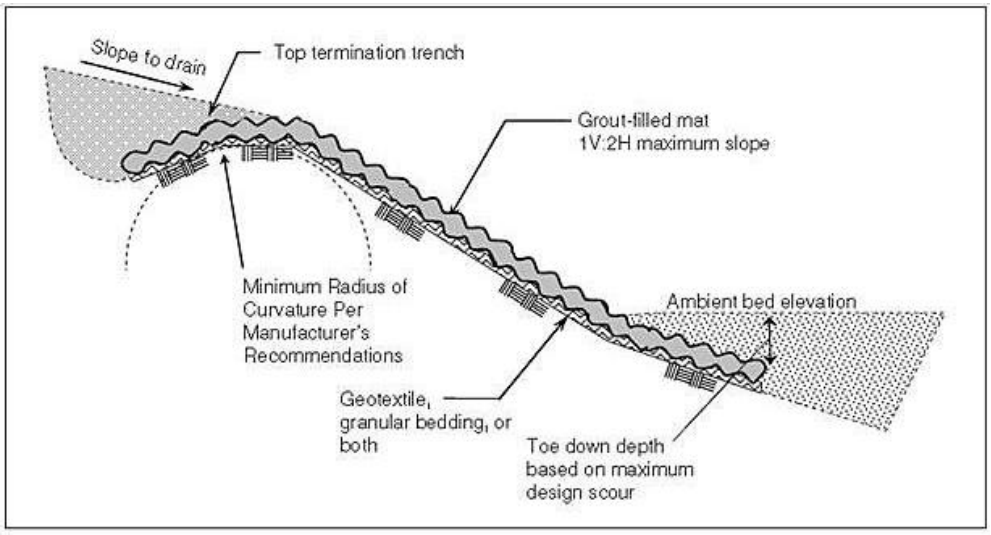
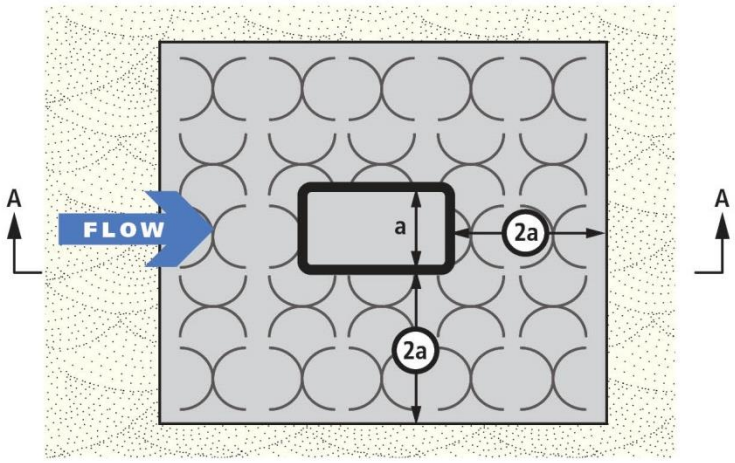


Figure 15.27 Suggested Installation Detail for Grout-Filled Mats Used as Bank Revetment



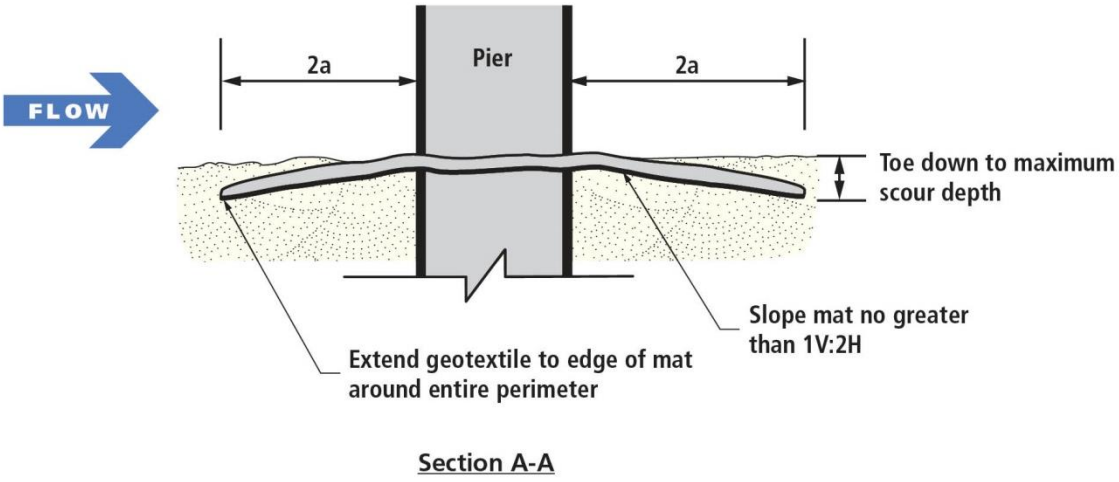


Figure 15.28 Layout for Grout-Filled Mats at Bridge Piers (Local Scour Prevention)

The grout should be designed to be pumpable with compressive strengths of at least 2,500 psi at 28 days. Admixtures are available to improve the flowability of the grout mix.

When stacked as forms for undermining void repairs, joints should be staggered between rows and anchored with rod dowels. Lower level bags should be permitted sufficient set time to support the succeeding vertical course. Grout injection should be performed in a manner to avoid bag rupture, prevent cold joints from forming, and to prevent discharge of grout material into the waterway. Void injection and venting pipes should be inserted during bag installation and positioned to ensure that the enclosed volume can be completely filled, and the enclosed water displaced. A four foot maximum spacing of injection/vent pipes is recommended.

15.3.4 Sheet Piling

Sheet piling can be effectively used to contain tremie concrete placements at foundations and as a countermeasure against stream migration (Figure 15.29), degradation (Figure 15.30), and local scour protection.

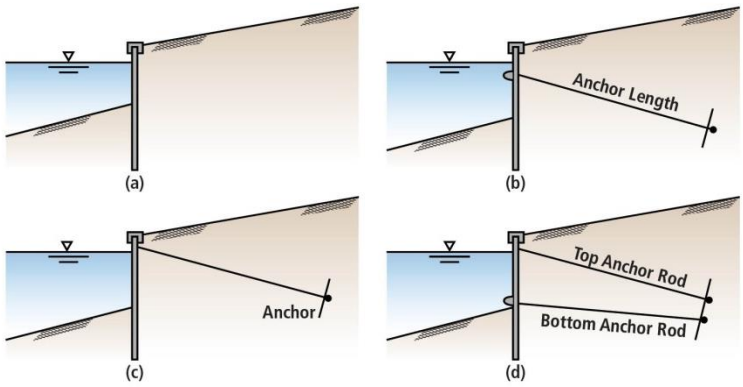


Figure 15.29 Anchorage Schemes for a Sheet Pile Bulkhead

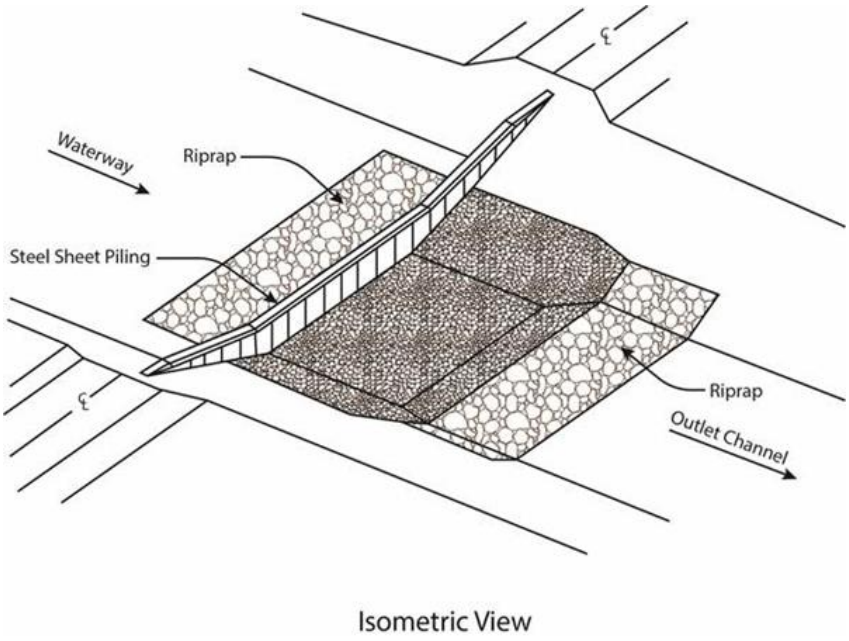


Figure 15.30 Isometric View of a Sheet Pile Check Dam

Sheet piling refers to the linear driving of adjacent pile elements into the existing soil to retain earth or prevent seepage. The piling can be constructed of interlocking steel, timber, or pre-cast concrete elements and can be configured with or without lateral anchorage. The pile elements are typically driven into the ground with crane mounted hammers, which are selected for the anticipated driving conditions.

Design of a sheet piling system requires knowledge of the existing soil conditions, evaluation of forces and lateral pressures, determination of the required penetration depth, anchorage selection and anticipated driving conditions. Pile elements should be structurally designed to resist the driving forces, surcharge loading, and the lateral earth forces encountered during typical and maximum anticipated scour conditions.

Selection of sheet piling solutions should consider overhead clearances for driving equipment, clearances to overhead and underground utilities, depth to impenetrable soil/rock layers and vibratory effects on nearby structures and facilities.

15.3.5 Articulated Block

Articulated block concrete (ABC) systems provide a flexible alternative to riprap, gabions, and rigid revetments. These systems consist of preformed units which either interlock or are held together with cables (or both) to form a continuous blanket. Example of the preformed unit shapes used in these systems are shown Figure 15.31. Typical uses include revetment and bed armoring, where the mat is placed across the entire channel cross section and for pier scour protection (see Figure 15.32 and Figure 15.33).

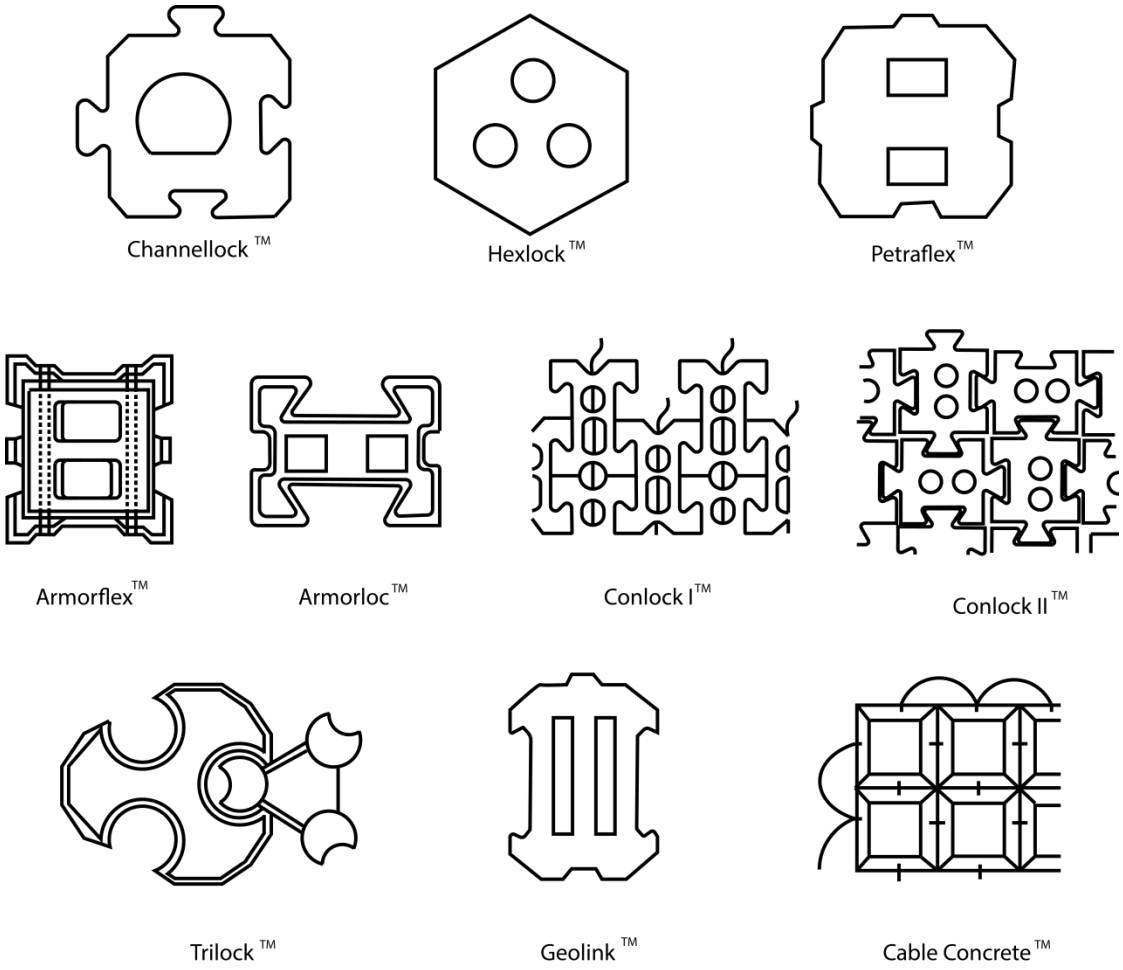


Figure 15.31 Examples of ACB Revetment Systems

Block systems vary in shape and size and will have unique design parameters suited to a range of anticipated hydraulic conditions. Block system should meet the physical requirements of ASTM D6684. In northern climates, the number of anticipated freeze/thaw cycles and corresponding weight loss cycles should be specified. Design and installation procedures should closely follow the manufacturer's recommendations to prevent uplift pressure and loss of subgrade soil through piping and liquefaction, which can lead to progressive plucking failure.

Articulated block systems are typically installed with a geotextile filter, although granular filtering can be used in certain situations. Installation in perennial streams will likely require stream diversions or cofferdams. Some systems maintain sufficient open spaces for the establishment of vegetation.

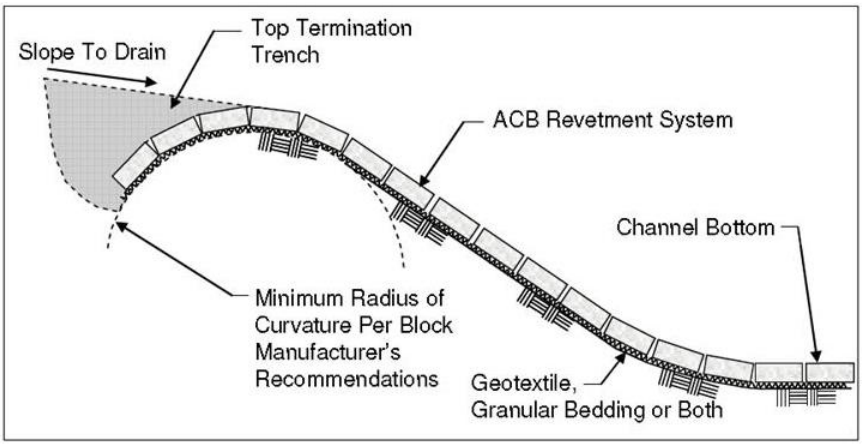

Slope To Drain
Top Termination Trench
ACB Revetment System
Channel Bottom
Minimum Radius of Curvature Per Block Manufacturer's Recommendations
Geotextile, Granular Bedding or Both

Figure 15.32 Recommended Layout Detail for Bank and Bed Armor

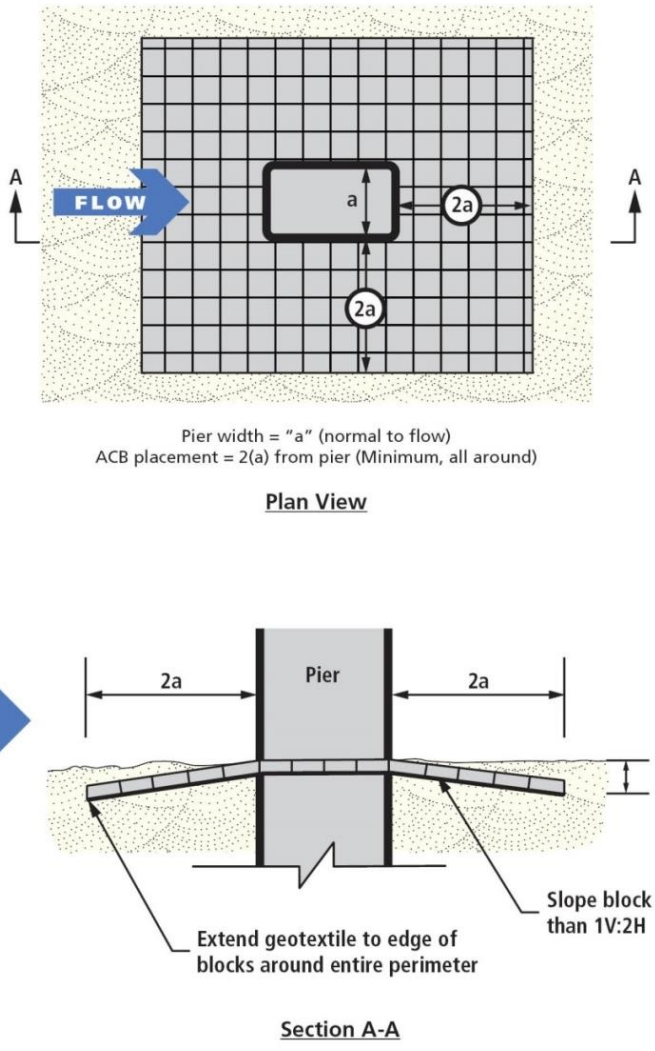


Figure 15.33 ACB Layout Diagram for Pier Local Scour Counter Measures

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Chapter 16 - Masonry Bridges

16.1 Common Types of Masonry Bridges and Masonry Bridge Elements

Masonry bridges are an early bridge type, and many are still in service. Many of these bridges have historic and cultural significance to the community and are on, or eligible for, the National Register of Historic Places. The intent of this chapter is to offer guidance on maintaining and preserving masonry structures.

16.1.1 Arches

Masonry arches come in three main types (i.e., coursed ashlar, random ashlar, and rubble) and can be constructed of either brick or stone.

16.1.1.1 Coursed Ashlar Masonry

Most brick arches and many stone arches are this type. Ashlar stones have neatly dressed edges. The joints are of a consistent thickness and blocks of the same size are used throughout, as shown in Figure 16.1. The horizontal joints are consistent over the length of the arch. To be considered ashlar masonry, the joints between the stones should be small and consistent. The joints can either be mortared or dry-laid.

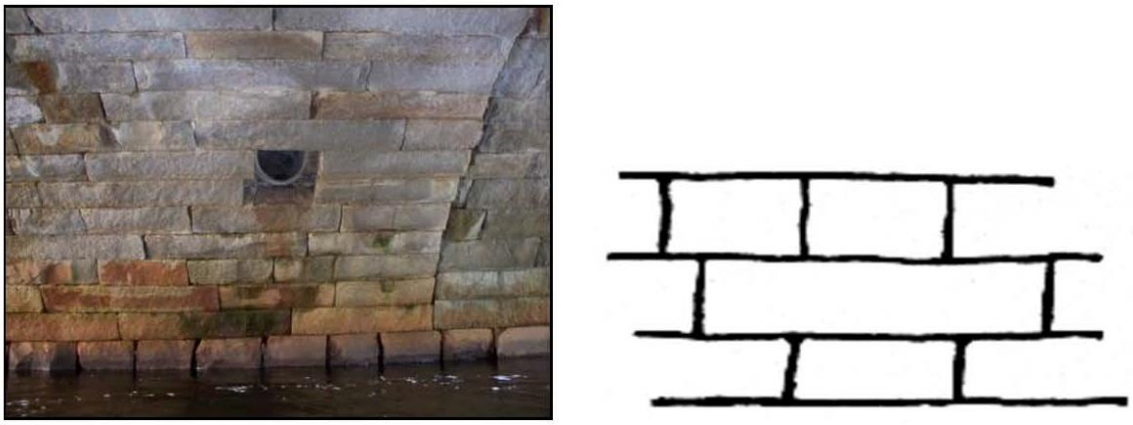

Figure 16.1 Coursed Ashlar Masonry (a) Photo and (b) Schematic

16.1.1.2 Random Ashlar Masonry

Random ashlar masonry is comprised of dressed stones or brick, where only the thickness of the masonry units and the longitudinal placement varies along the arch ring (arch stone) or wall face (spandrel), as shown in Figure 16.2. This type of construction is more common in vertical walls (e.g., substructure units) than in arches.

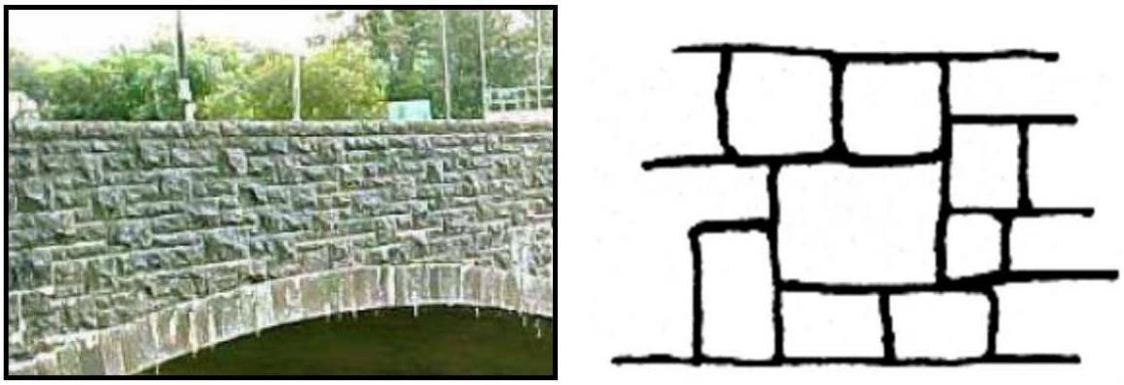

Figure 16.2 Random Ashlar Masonry (a) Photo and (b) Schematic

16.1.1.3 Rubble Masonry

Rubble masonry is comprised of undressed or roughly dressed stones. Figure 16.3 shows squared rubble masonry along the face stones (spandrel wall). The interior of the arch (intrados) in Figure 16.3 is constructed of stones that have not been squared and are placed in random coursing. Rubble masonry can be coursed or random, similar to ashlar masonry. It can also be grouted or dry-laid.

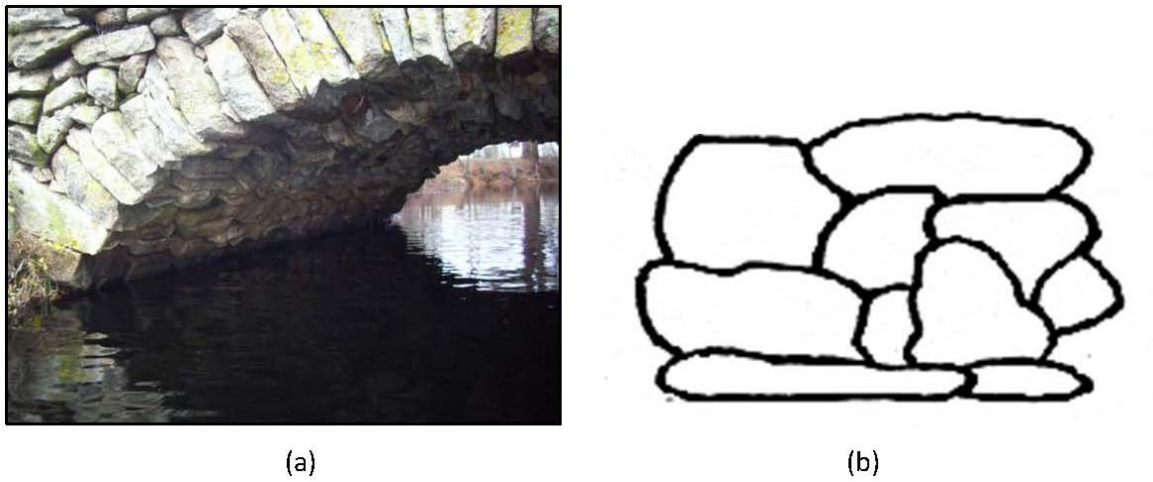

Figure 16.3 Rubble Masonry (a) Photo and (b) Schematic

16.1.1.4 Arch Elements

Figure 16.4 defines most of the terminology required for the discussion of stone arches. The interior of the arch barrel is called the intrados and the exterior is called the extrados. The spring line is the horizontal line connecting the furthest apart points on the intrados. The arch rise is the distance from the spring line to the underside of the keystone. Not all stone arches have a keystone. For an arch without a keystone, the crown is common terminology for the top of the arch.

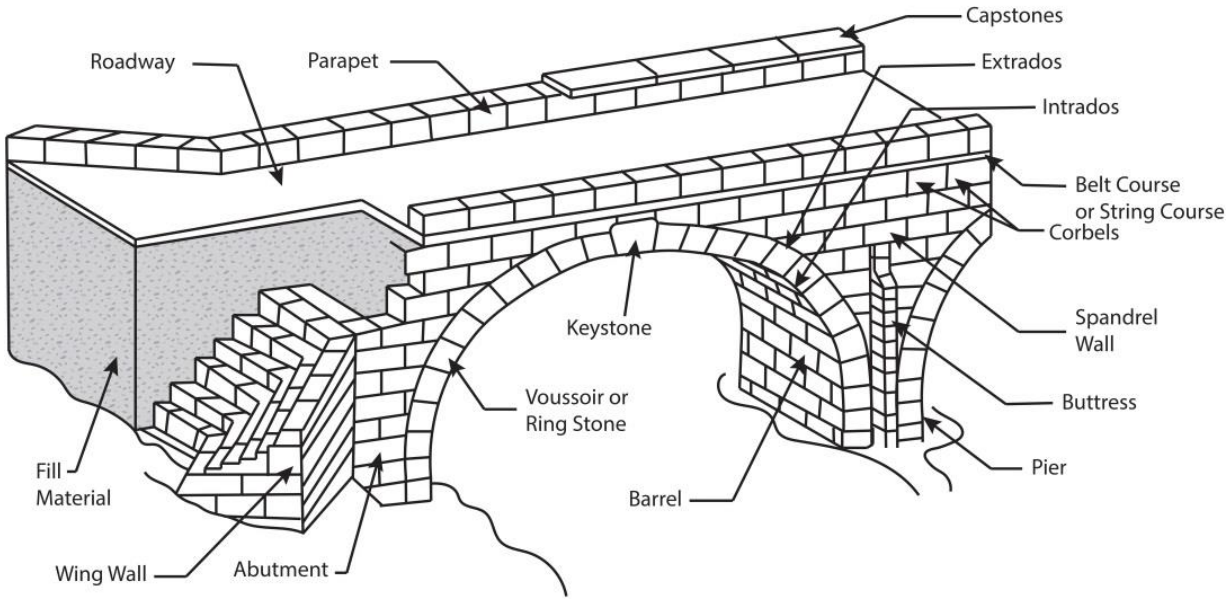

Figure 16.4 Arch Elements

Arches typically rely on the soil to provide lateral stability and to distribute the live load among the various elements. Typically a 1 foot wide section is assumed by engineers for structural analysis purposes and the soil is assumed to distribute the live load to the arch. However, if there is less than 2 feet of fill over the structure, the soil cannot be used to distribute the fill. In cases of inadequate fill depth, the length of the stones themselves becomes critical in determining the transverse width that the live load can distribute, making the minimum stone length a critical dimension in determining the structural capacity of the masonry structure.

A masonry arch is made from identical wedge-shaped pieces known as voussoirs (the masonry units). The arch is built on falsework (a temporary supporting framework) since it cannot stand until the last stone is in place. Once complete, the falsework is removed and the arch at once starts to thrust at the abutments. Inevitably the abutments will give way slightly and the arch will spread.

Figure 16.5, greatly exaggerated, shows how the arch accommodates itself to the increased span. The arch has cracked between voussoirs – there is no strength in these joints, and three hinges have formed. The arch is not, however, on the point of collapse – the three-hinge (also known as the three-pinned) arch is a well-known and perfectly stable structure.

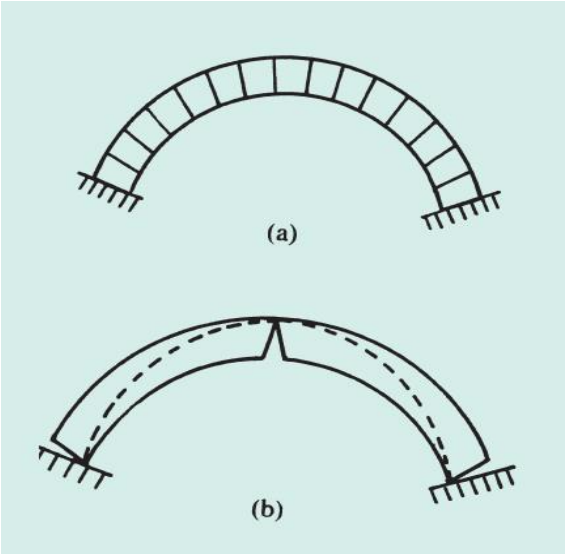

Figure 16.5 Arch Accommodation to Increased Span

If the arch is backfilled (the soil over the arch is placed) prior to the formwork being removed, then the arch will perform as one homogeneous structure (no hinges will form in the arch). The lateral earth pressure as well as the vertical earth load provide resistance to the arch thrusting at its abutments. Therefore, the arches ability to spread is severely reduced. In this case, the arch would be modeled with fixed connections at the abutments and as a homogeneous mass (although the arch ring thickness may vary) throughout the arch itself.

Both the fixed arch and the three-pinned arch are valid models for masonry arches. While the supports for a concrete or steel arch can be modeled with pinned supports (and without a hinge at midspan) this model is not valid for masonry structures as it would make the structure unstable. Additionally, the hinge at the top of the arch is almost never centered on the arch due to the presence of the keystone. The keystone is typically a wedge shaped stone that is larger than the surrounding stones. The purpose of the keystone being large is that it will be able to displace into the arch to accommodate the gap caused by the formation of the hinge at the crown.

It is sometimes difficult in the field to determine which type of arch construction was used, and plans and specifications are not typically available either. Most masonry arches will show cracking in the grout and many masonry arches do not even use grout. The most common method of analyzing these structures is to assume the masonry arch is fixed at the supports.

Typically arches that have a keystone or are an approximate half circle are modeled as fixed at the supports and the arches without keystones and flatter arches are modeled as three-pinned arches so that the passive pressure can be used to help resist the thrust of the arch.

In order to analyze an arch, careful field measurements are required. The springline, rise and shape of the arch are critical to getting an accurate representation of the arch and henceforth an accurate load carrying capacity.

16.1.2 Masonry Slab Bridges

Masonry slab bridges are typically small span structures, less than 10 feet. An example of a granite slab bridge is shown in Figure 16.6. In general, masonry slab bridges have not been

constructed since the early 1900's. These structures are comprised of butted individual slabs whose ends are resting on the substructure. The width of the slabs can vary over the width of the bridge. The thickness of the slabs is typically consistent over the entire structure, however, if for some reason the thickness varies, the least thickness should be used to analyze the bridge.

Figure 16.6 Granite Slab Bridge

16.1.3 Substructure Elements

Abutment and piers can be constructed of masonry elements. The masonry units can be solid throughout the elements, or in areas where production concrete is not readily available, rubble abutments are a viable alternative. The rubble abutments are formed by dumping large quantities of masonry rubble (uncut stones, brick or even concrete rubble). The structure is then typically coated with concrete to form a cohesive structure. Another common use of masonry units in substructure is to use the masonry to form an aesthetic veneer over a reinforced concrete structure. It is common to find concrete caps and extensions on masonry substructure elements (as shown in Figure 16.7). Sometimes rebar is drilled and grouted into the masonry to attach the concrete elements; however, the concrete can also be cast against the stone with no mechanical connection provided.

Figure 16.7 Masonry Substructure

16.1.4 Brick, Stone and Mortar Characteristics

Brick

Brick is manufactured by forming clay or shale into blocks and hardening it by heating. Brick can be either molded or extruded and can vary greatly in appearance and composition. While there are standard brick sizes, bricks can also be manufactured in custom sizes and with custom finishes. Brick can be glazed to provide a different finish and to increase protection from the elements. An example of a brick arch and spandrel walls is presented in Figure 16.8.

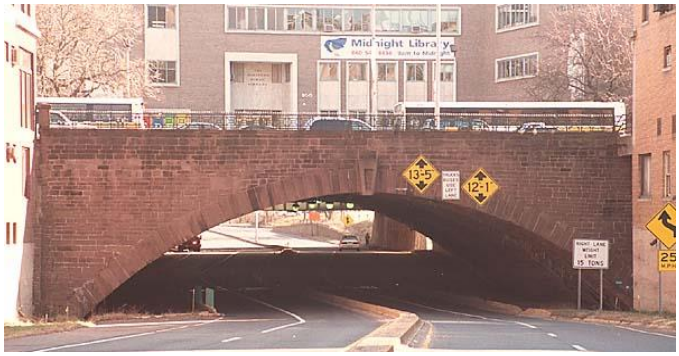

Figure 16.8 Brick Arch and Spandrel Walls

There are various ASTM standards for brick. ASTM C62 is the Standard Specification for Building Brick, ASTM C67 is the Standard Test Method for Sampling and Testing Brick and Structural Clay Tile, and ASTM C216 is the Standard for Facing Brick. There are 2 grades of brick commonly used in bridge construction. Grade SW is specified for severe weathering and also is the default type if the grade of the brick is not specified. Grade MW is specified for moderate weathering and is typically not used except in the southernmost sections of the United States.

The grades of brick can be further broken down into types. If no type of brick is specified, typically Type FBS is used. Type FBS brick is applicable for general use in masonry construction. If a higher degree of precision is required to accommodate a lower permissible variation in size, than Type FBX is required. Type FBA is specified for general use where characteristic architectural effects resulting from non-uniformity in size and texture of the individual units is desired. While there are other types of brick available, those listed above are the primary choices for masonry construction. Comparison of Type FBS and Type FBX is shown in Table 16.1.

Table 16.1 Type FBS and Type FBX Bricks

Brick Type	Waste	Chips	Cracks	Size Variation from Each Other	Warpage
FBX	Up to 10 percent	1/8 inch edge 1/4 inch corner	Any not seen from 15 feet away	5/16 inch from each other	1/16 inch for under 8 inch dimension
FBS	Up to 20 percent	5/16 inch edge 1/2 inch corner	Any not seen from 20 feet away	1/2 inch from each other	3/32 inch for under 8 inch dimension

Stone

Similar to brick, stone bridges can be constructed of several different types of stone. Igneous stone (e.g., granite), sedimentary stone (e.g., limestone or sandstone) and metamorphic (e.g., marble) are the main categories for stone. Sedimentary rock typically has little flexural strength and is therefore not used in slab bridges. All three stone categories are applicable for use in compression applications. Geological maps which show the typical bedrock profile for the area that the bridge is located are helpful in determining the type of stone used in a historic bridge. However, without testing or cleaning it is difficult to determine the stone type, as weathered stone typically has a gray appearance regardless of composition. Below is a chart with basic information on various types of stone that may be helpful in rehabilitation and repair work.

Table 16.2 Basic Information on Various Stone Types (NHI Course 134062)

Stone Type	ASTM Standard or Test Method	Absorption by Weight (percent) ASTM C 97	Density (lb./cf): ASTM C 97	Modulus of Rupture (psi): ASTM C 99	Compressive Strength (psi): ASTM C 170	Flexural Strength (psi): ASTMC 880
Marble	ASTM C 503	0.75	144 to 175	1000	7500	1000
Low-density limestone	ASTM C 568	3.0	110	400	1800	n/a
Med-density limestone	ASTM C 568	7.5	135	500	4000	n/a
High-density limestone	ASTM C 568	12.0	160	1000	8000	n/a
Granite	ASTM C 615	0.4	160	1500	19,000	1200
Sandstone	ASTM C 616	1.0	125	350	4000	n/a
Quarzitic sandstone	ASTM C 616	3.0	150	1000	10,000	n/a
Quartzite	ASTM C 616	8.0	160	2000	20,000	n/a
Slate	ASTM C 629	0.25	n/a	7200 to 9000	n/a	n/a
Travertine	ASTM C 1527	2.5	144	1000	7500	1000

Mortar

Mortar is a mixture of cement, lime, sand and water that is installed while still in a plastic state. The mixture then cures (hardens) to a solid state. Mortar is used to separate the masonry units and allows for a construction tolerance without sacrificing contact between the stones. The mortar also helps to bond the masonry units to each other and prevents water from seeping between the masonry units. The mortar should be considered sacrificial to the masonry components and designed to deteriorate sooner that the masonry. (The mortar should have a lesser strength than the masonry units that it joins.) Mortar should be designed to be more permeable than the adjacent masonry to allow water vapor to escape from the masonry units.

There are several ASTM standards for mortar. They include C270 Standard Specification for Mortar for Unit Masonry, C780 Standard Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry, and C1324 Standard Test Method for Examination and Analysis of Hardened Masonry Mortar.

Mortar is comprised of binders, aggregates, and water. Binders typically include Portland cement, lime and other manufactured or natural cements. The aggregates in mortar are sand and pigments if powdered pigments are used to vary color or texture. Masonry structures can be built without mortar. Mortar used in bridges that were built prior to the 1800's is typically either an earth/clay mortar or a sand/lime mortar. Portland cement mortars were not in use until the 1870's. Masonry cement (Type N, M, S & O mortars (referenced in the MBE and ASTM standards) has been used since the 1930's and ready mix mortars have been available since the 1980's.

There are several characteristics of mortar that need to be considered when designing the mix for a masonry arch. The compressive strength of the mortar should always be less than the masonry units that it is connecting. The amount of lime in the mix adjusts the strength, as well as other attributes. Increasing lime content corresponds with decreasing strength of the mix. The mortar is intended to fail before the masonry units. Mortar can be repaired; however, cracking of the masonry units is a much more involved repair.

The tensile strength of the mortar is another design consideration. It is not uncommon (although it is also not desirable) for part of the arch to be in tension. Typically this occurs at the extreme limits of the arch barrel depth, outside of the kern. The kern is defined as the middle third of the arch barrel. In cases where the combined dead and live load thrust falls outside of the kern, sections of the arch are in tension and do not contribute to the load carrying capacity of the arch. Figure 16.9 (right) visually illustrates potential areas where tension can occur due to applied load (shown by red arrow). This sketch is greatly exaggerated to illustrate the point. Imagine a very heavy load passes over the stone arch with such force the arch flattens out. It can still be stable, but the load was so large that it placed portions of the arch in tension (gaps between stones in Figure 16.9 right).

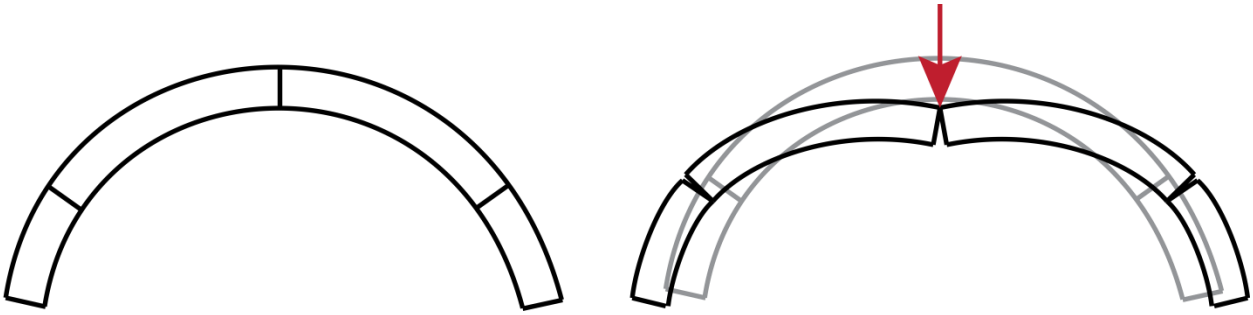

Figure 16.9 Potential Areas of Tension in an Arch

The intent is to provide a mortar that will bond the masonry units to each other if tension is present in the arch. Therefore, cohesion and bond strength of the mortar is another design consideration.

To avoid damage to mortar due to road surface runoff seepage, the porosity of the mortar is designed to be higher than that of the adjacent masonry units. It is important that the mortar accommodates the moisture before the masonry units can absorb it. If the water does seep into the mortar, the durability of the mortar also becomes another consideration. The freeze thaw cycle of water can be detrimental to mortar joints. It can cause cracking and spalling of the mortar in the joints. However, the mortar should still be designed to fail prior to the masonry units.

Several other materials can be added to mortar to adjust its characteristics. Pigments or colorants can be added to adjust the color of the mortar. One common use of pigments is to darken the mortar. This is performed on rehabilitation projects where a previously un-mortared arch requires mortar. The mortar is placed in the joints and then raked back so that the darker mortar is in the shadow of the stones and is therefore less visible. Other common admixtures include bonding agents, masonry cement components, and water repellants. However, on rehabilitation jobs, the use of water repellants in the mortar should be carefully considered so that the porosity of the stone is always exceeded.

Table 16.3 shows the typical mortar types, their compressive strengths, and their typical uses. The schematic in Figure 16.10 explains the benefits of soft mortar versus hard mortar. Soft mortar is considered to be “self-healing”, but it has limited compression capacity. Hard mortar is often high compressive strength but brittle. Please refer to Figure 16.10 for further discussion of the benefits soft mortar over hard mortar.

Table 16.3 Mortar Types and Descriptions

Mortar Type	Minimum 28-Day Compression Strength	Description and Use
M	2500 psi	High compressive strength and durability for below-grade applications
S	1800 psi	High bond strength for flexural conditions
N	750 psi	For exposed masonry above-grade
O	350 psi	Soft masonry units

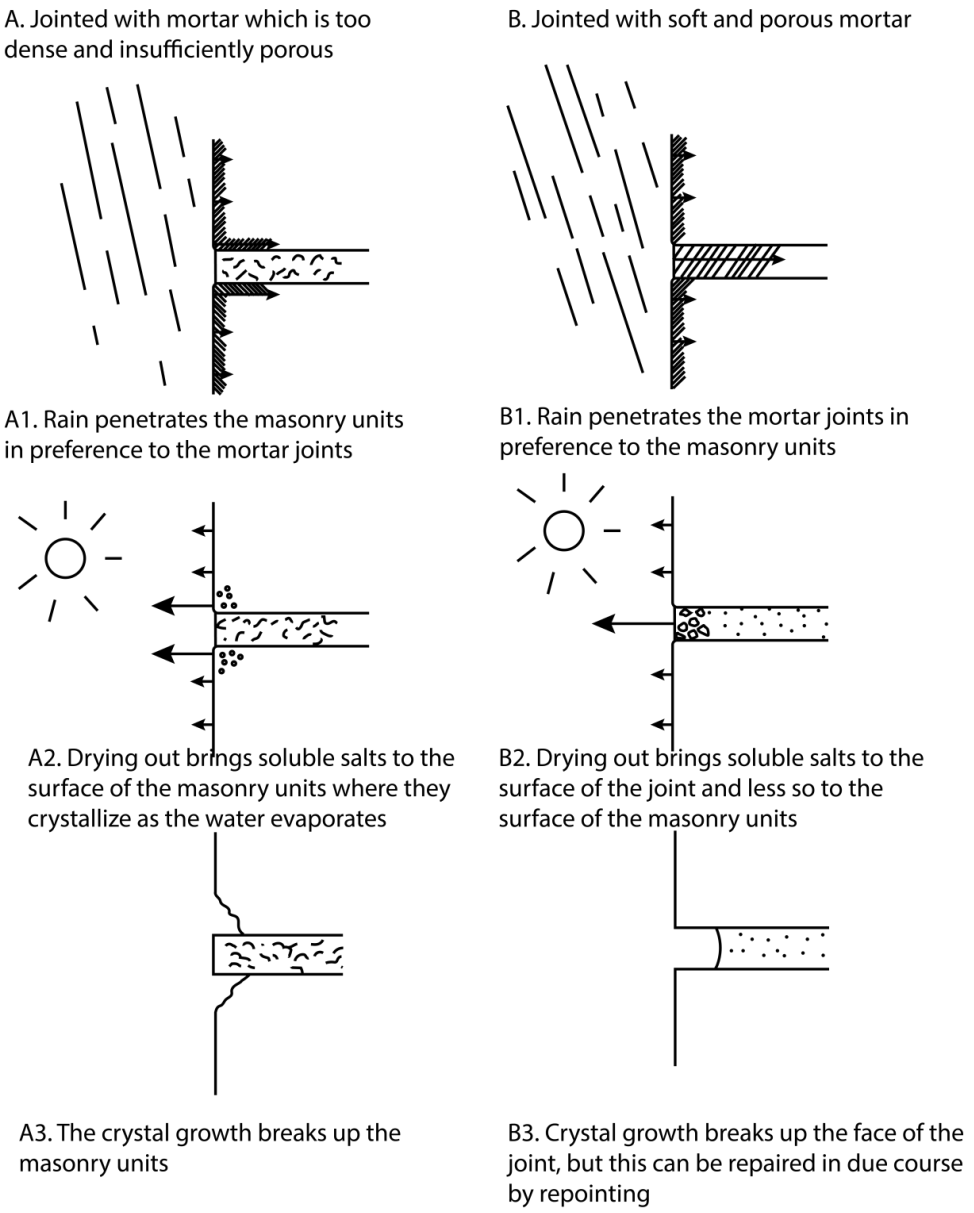
A. Jointed with mortar which is too dense and insufficiently porous
B. Jointed with soft and porous mortar
A1. Rain penetrates the masonry units in preference to the mortar joints
B1. Rain penetrates the mortar joints in preference to the masonry units
A2. Drying out brings soluble salts to the surface of the masonry units where they crystallize as the water evaporates
B2. Drying out brings soluble salts to the surface of the joint and less so to the surface of the masonry units
A3. The crystal growth breaks up the masonry units
B3. Crystal growth breaks up the face of the joint, but this can be repaired in due course by repointing

Figure 16.10 Benefits of Hard and Soft Mortar

16.1.5 Architectural Veneer

Architectural veneers are common in bridges. They are used for aesthetic purposes and typically are not a structural element. They are used to cover the structural element in order to form a more pleasing appearance.

Veneers can be mechanically attached by anchors or they can be cast into the supporting elements (see examples shown in Figure 16.11). When they are cast into the supporting element, the only element holding the masonry in place is the bond to the backing element. There is also no way to install drainage. Therefore, use of mechanical anchors is the preferred method to attach masonry veneers.

Figure 16.11 Masonry Veneer Showing Strap Anchors (left) and Under Construction (right)

Masonry anchors are widely variable. Dovetail anchors and masonry strap anchors are both common types (see Figure 16.12). One side of the anchor is attached to the concrete typically by a mechanical connection. The other side of the anchor is placed into the mortar bed between stones. Once the mortar dries, the anchors provide a mechanical connection between the reinforced concrete structural wall and the masonry veneer. An air space can be left between the reinforced concrete wall and the veneer through the use of anchors. The void between the two layers can be used to provide drainage.

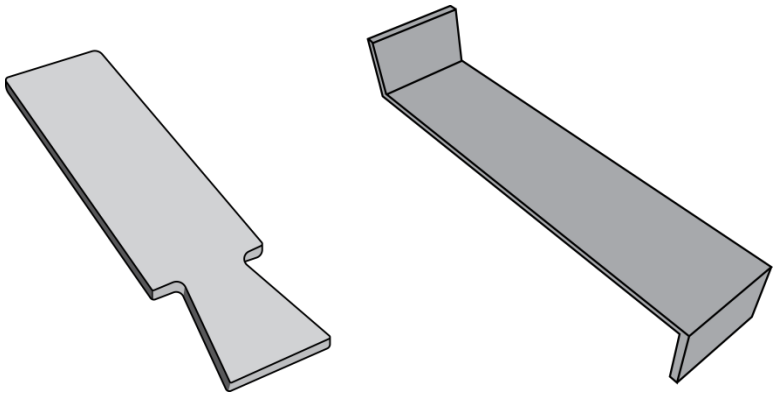

Figure 16.12 Masonry Anchor Types. Dovetail Flat Anchor (left) and Strap Anchor (right).

16.2 Preventative and Basic Maintenance of Masonry Bridges

16.2.1 Inspection

The Federal Highway Administration's (FHWA) National Bridge Inspection Standards (NBIS) require that all bridges be inspected on a two year or lesser interval unless approval from the FHWA is attained for exceeding the two year requirement for a particular bridge.

Inspection and proper documentation of masonry bridges is needed to ensure there are no signs of potential distress and to identify maintenance and preservation needs. Items to look for include:

What To Look For

- Efflorescence
- Splits and spalls of mortar and masonry units
- Deteriorated mortar, cracking in mortar and in masonry units
- Slipped masonry units
- Displacement of masonry
- Scour
- Vegetation growing in masonry
- Evidence of water infiltrating through the masonry
- Impact damage

16.2.1.1 Signs of Potential Distress

Efflorescence

Efflorescence, when present, is indicative of soluble salt migration into the masonry structure. The salt laden water typically seeps into the mortar joints first; therefore water stain and efflorescence is most likely to be first visible in the mortar joints. However, if efflorescence is visibly emerging from the masonry units, the area should be monitored for future deterioration.

Split and spall

Splitting and spalling of the mortar joints is anticipated and can be repaired by repointing if required. If the porosity of the masonry unit exceeds that of the mortar, or if the mortar is harder than the adjacent masonry units, spalling of the masonry units can occur. Both conditions should be monitored and appropriate corrective action should be undertaken when required by the field condition.

Mortar Deterioration

Mortar deterioration is expected to occur over the service life of the bridge. The deterioration itself is not necessarily an issue. However, if the contact between the stones is significantly reduced, if water infiltration is occurring, or if other signs of distress are present, repair of the joint may be required. An example of mortar deterioration is shown in Figure 16.13.

Figure 16.13 Mortar Deterioration (indicated by arrow)

Cracking

Cracking in the mortar is common and is not necessarily a sign of distress for the masonry structure. The mortar is designed to fail in order to alleviate the pressure on the masonry itself. However, if there is cracking in the masonry, there is a condition that needs to be monitored. An example of a crack in masonry is shown in Figure 16.14. Many cracks in the mortar and masonry occur either when the forms are removed or while the arch is being backfilled. These cracks should be noted, but if the condition of the crack (width or length) is not changing, there is most likely no cause for concern. However, if the crack monitoring is noting differences, the crack should be closely monitored and changes in condition should be assessed. Having an engineer review the crack and its effect on the load carrying capacity of the bridge is highly recommended.

When to Call the Engineer

Call the Engineer when an existing crack changes in appearance, either in width or length.

Masonry arches are typically load rated based on an assumed one foot unit width. Any cracks that create segment of masonry smaller than one foot should be noted.

Any cracks in the masonry of a slab bridge should be monitored. Transverse cracks indicate that failure has already occurred. Transverse cracks require immediate attention and shoring or road closure over the cracked slab should be considered, at least until an engineer can make an assessment. Longitudinal cracking in the masonry of slab bridges is not as critical. Each individual slab acts as a unit and the load is distributed over the width of the slab unit. A longitudinal crack reduces the distribution width available and is therefore important to note, but not necessarily critical to the load carrying capacity of the bridge.

Figure 16.14 Transverse Crack in Stone Arch

Slipped Masonry Units

A slipped masonry unit is a stone or brick that is lower than the adjacent stones and one that has displaced below the interior of the arch barrel (or the intrados). Examples of a slipped masonry units are presented in Figure 16.15 and Figure 16.16. Many times the stone slips during construction. This typically only occurs in un-mortared masonry arches and as brick arches are almost always mortared, it is prevalent mainly in stone arches. During a flood event, the fines and chinking stones (small stones used to fill voids between adjacent masonry units) can be washed away causing the stones to loosen in the arch ring. Therefore, monitoring of slipped stones is important after flooding. The slipped stone should be noted and monitored. An engineer should be notified if any change in the condition of the slipped stone is noted. However, the presence of a slipped stone is not typically critical to the load carrying capacity of the structure. Figure 16.15 shows multiple slipped stones, which resulted in arch deformation, this condition is more critical than a single slipped stone.

Figure 16.15 Multiple Slipped Stones

Figure 16.16 Single Slipped Stone

When to Call the Engineer

If masonry displacement is observed, the Engineer should be called prior to beginning repairs.

Missing Masonry Units

One modification historically made to masonry bridges is the removal of masonry units for the installation of utilities or drainage. Typically, the arch (or the soil in a slab bridge) is able to create an "arching effect" over the missing masonry units. The condition should be noted, but is not typically critical to the load carrying capacity of the bridge. An example of missing stones at a drainage pipe is shown in Figure 16.17.

Figure 16.17 Missing Stones in a Masonry Arch

Masonry Displacement

Masonry displacement, or arch deformation, is an indicator that failure of the masonry arch has occurred. Typically the deformation is localized, but the arch requires immediate attention. Barriers to keep traffic away from the deformed section or temporary shoring may allow sections of the arch to remain in service, but repair should be given priority. An example of masonry displacement is presented in Figure 16.18.

Figure 16.18 Arch Deformation caused by Masonry Displacement

Scour

One sign of potential distress, especially in arches, is scour. Scour is the erosion of the channel bed. Scour is critical if it occurs adjacent to the footings. If the soil behind the arch is scoured away, the integrity of the arch is compromised. The soil behind the arch is used to resist the thrust from the arch and prevent the arch footing from kicking out. As many masonry bridges do not have mortared stones, scour holes can provide a weakness in the foundation that allows the foundation to displace, therefore causing a displacement in the arch or and slab bridge foundation. Undermining can also occur to the foundation elements if scour is allowed the go unchecked.

Stream beds are dynamic and it is not uncommon for scour holes to form during high flow condition and then be filled back in by material suspended in the water settling back out during normal flow. If silt deposits are found during wading, probing and/or underwater bridge assessment, further investigation may be required to determine if a scour condition is present.

Scour should be closely monitored or repaired depending on its severity.

16.2.1.2 Condition Assessment

Visual Inspection

Visual inspection is very important for masonry bridges, as most of the potential signs of distress can be discovered with a visual inspection. During the inspection, it is also important that vegetation be removed or noted for future removal.

Wading/Underwater Inspection

Underwater inspection is used to determine stream bed erosion. Scour, and eventually undermining of the footings, can occur if the erosion is allowed to continue. Scour monitoring charts which document the stream bed condition over time are an important tool for inspectors. Scour monitoring charts consist of stream bed profiles taken upstream and downstream of the bridge and in other critical locations to monitor any changes in the stream bed profile.

Test Pits

Test pits are recommended for masonry structures for condition assessment and composition determination. As masonry structures are typically old, it is uncommon for plans to be available. If adequate determination of the structure cannot be made from below, then a test pit is recommended. Masonry units are sometimes used as a veneer over a reinforced concrete arch. Without the test pit, the inspector would have limited information to determine what the structural components actually are. The thickness of the arch ring is another component that a test pit is used to discover. Often times, the outside ring of the arch is decorative and may not be made of the same material or thickness as the interior part of the structure. Test pits are also used on masonry substructures to determine the limits and the presence of a batter. A batter is a vertical slope to the masonry structure that has a horizontal component. Probes, borings and cores can also be used to determine the extents of an existing masonry substructure. An example of a test pit is shown in Figure 16.19.

Figure 16.19 Test Pit

As the top of the masonry structure is typically exposed to more salt (that migrated from the roadway above) than the underside, the test pit can also be a way to determine the condition of the extrados of an arch bridge or top of a buried slab bridge. If efflorescence is present on the underside, test pits to expose the top should be considered.

Inspection Openings and Cores

Masonry arches are sometimes comprised of multiple layers. Sometimes the layers are all of the same material, but other times the material visible from the outside is only a veneer. Inspection openings can be used to determine the material composition of the structure. The

inspection opening is created so a visual assessment and measurements can be made. The opening is then patched. An example of an inspection opening is presented in Figure 16.20.

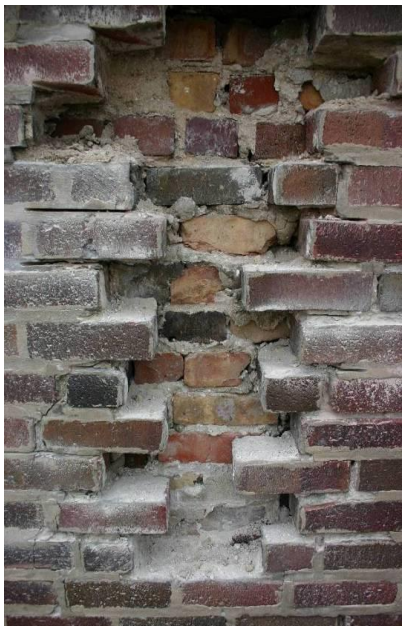

Figure 16.20 Inspection Opening

Cores can also be used to determine the thickness and the material composition of a masonry structure. If the core is taken along the joint of masonry units, the contact area can be determined. Cores can be tested to determine the strength of the masonry units and the degree of contamination (chlorides, etc.) in the masonry if applicable. An example of core removal is shown in Figure 16.21.

(a) (b)

Figure 16.21 Masonry Cores (a) Removing the Core and (b) Finished Masonry Core

Non-destructive Testing

Non-destructive testing can also be used to determining the composition and quality of the masonry structure. Typical methods for non-destructive evaluation of masonry structures include impact echo testing and ground penetrating radar, as described below.

Impact echo testing is a common method of evaluation for both concrete and masonry structures. Impact echo testing uses impact generated sound/stress waves. The waves pass through the structure until they are interrupted by either internal flaws in the structure or by

the structure ending. Impact echo can be used to determine the thickness of the masonry unit. Impact echo can also be used to find cracks, delaminations, voids, or other deficiencies in the masonry structure. Impact echo testing principles and in practice are shown in Figure 16.22.

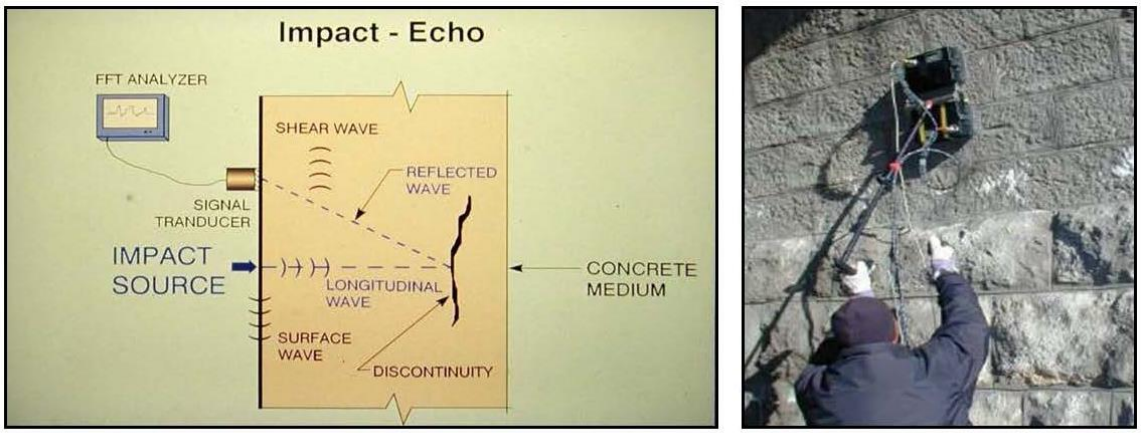

Figure 16.22 Impact Echo Testing (a) Schematic and (b) In Practice

Ground penetrating radar is another common method of non-destructive testing for masonry bridges. Ground penetrating radar is similar to impact echo except that it uses electromagnetic radiation, as opposed to sound waves, to generate its data. Ground penetrating radar can also be performed from the ground surface where impact echo requires application of the impact to the masonry structure. An example of ground penetrating radar application is presented in Figure 16.23.

Figure 16.23 Ground Penetrating Radar Testing

16.2.2 Maintenance

Removal of Foreign Material / Vegetation

One of the most important maintenance procedures on masonry bridges is the removal of foreign material and vegetation. Dirt and debris can trap moisture and accelerate the freeze / thaw deterioration cycles. Vegetation also causes problems with the continued performance of

the structure. The roots are destructive as they can grow through the structure in their search for water. They can cause displacement of the stones. Additionally, when trees fall, they can further damage the masonry structure. An example of vegetation growing through masonry joints is presented in Figure 16.24.

It is also important that vegetation and foreign material be removed from the stream, if applicable. This material can cause the stream to re-route causing potential scour and undermining problems. Additionally, the foreign material can cause the stream to flow at a higher elevation which can erode the mortar (or fines) between the stones. The water can also overtop the roadway, causing displacement of the spandrel walls as well as accelerated deterioration to the structure.

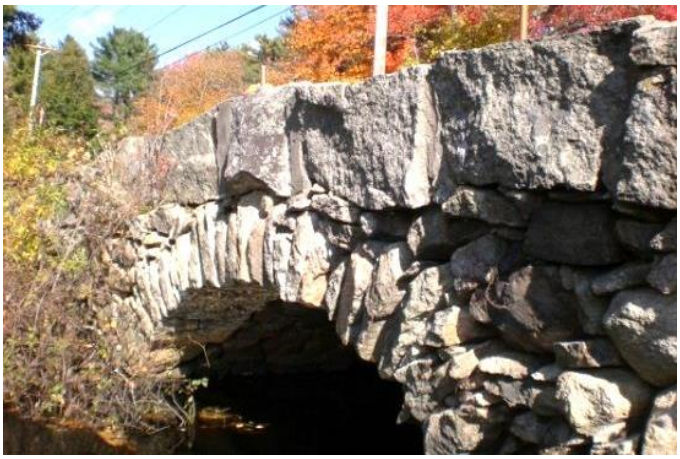

Figure 16.24 Vegetation Growing Through Masonry Joints

Cleaning

Cleaning of masonry structures is important to remove efflorescence and soluble salts from the surface of the masonry units and mortar. Cleaning can also be an important tool in determining the type of stone used in the masonry structure. Over time, most stones weather to a gray color similar to that of granite. By cleaning the stones, the true surface of the stone can be exposed. It is recommended that several stones are cleaned (a wire brush and water are typically adequate) and assessed prior to determination of material composition. Low pressure power washing (less than 400 psi) is also acceptable, however the use of high pressure water is not recommended as the high pressure water (over 400 psi) can actually damage the masonry structure and the mortar joints. An example of cleaning with low pressure water is shown in Figure 16.25.

Figure 16.25 Cleaning with Low Pressure Water

Arresting Water Infiltration

When water seeps from the surface of the roadway to the masonry structure, it brings salt and other contaminants with it. These contaminates can shorten the surface life of the structure. Cleaning out adjacent drainage structures, removing debris that may be impeding the travel of runoff from the bridge, and patching the pavement are all routine maintenance procedures that can increase the service life of the masonry structure. A possible result of water infiltration is shown in Figure 16.26.

Figure 16.26 Evidence of Water Infiltration

16.3 Repair and Rehabilitation of Masonry Bridges

The intent of repair and rehabilitation of masonry structures is to extend the service life of the bridge, correct a deficient condition, or to bring the bridge up to current standards (such as changing the crash barrier). As many of the structures are historic, it is important to match the existing structure or to repair it in a way that is sensitive to the historical significance. However, public safety is still the first priority.

16.3.1 Repointing of Mortar

Mortar is a sacrificial element of a masonry bridge, and it is anticipated that it will need to be repointed over time. However, keeping the mortar in good repair is an essential aspect of extending the life of a bridge.

Depending on the age of the structure and the preferences of the designer or mason, a variety of binders could have been used in the original construction. The type of binder significantly affects both the characteristics and the performance attributes of the mortar. For example, mortars made with lime binders tend to have a lower compressive strength, lower bond strength, and greater permeability than mortars made with Portland cement as the binder. Mortars with a high lime content also have the ability to re-seal small hairline cracks. This allows the mortar to provide greater resistance to rain penetration and allows the moisture to escape after it has penetrated. However, the high lime content also delays curing of the mortar and the time it will take for the mortar to reach full compressive strength.

When designing the mortar mix, it is very important that the new mortar have a lower compressive strength than the units that it will be connecting (as previously mentioned). The mix must have a greater porosity than the adjacent masonry units. As not all of the mortar is removed in the joints, it is also important that the proposed mortar be compatible with the existing mortar. It is recommended that the mortar be replaced only if there is evidence of deterioration. Therefore, it is also important that the proposed mortar have a similar cured appearance to the existing mortar, especially if aesthetics are a concern.

Recommended repair method is as follows:

Suggested Procedure

Mortar Repointing

1. It is recommended that 2 to 2-1/2 times the thickness of the mortar or a minimum of 3/4 inch (whichever is greater) be removed for repair. This will ensure the replacement mortar has adequate bond area. The mortar should be removed using hand tools or a small pneumatic chisel. The use of power tools such as a grinder is not recommended because it is difficult to control and can result in damage to the adjacent masonry units. Square cuts at the limits of the removal are recommended. Figure 16.26 demonstrates the recommended mortar removal.
2. Debris should then be removed from the joint. This can be accomplished using a nylon brush, low pressure compressed air (40 to 60 psi) or low pressure water (less the 400 psi). Care should be taken during the removal of the debris to ensure that additional mortar depth is not removed by the debris removal process.
3. Any loose stones should be carefully removed, cleaned, and re-set prior to repointing.
4. Mortar used for repointing should be stiff, as stiff mortar will shrink less as it cures. Mortar should not be allowed to dry out during use and should be reworked at intervals prior to being placed in the joints. Retempering (or adding additional water) to the mix is not allowed. Repointing should not be done if the temperature is below freezing. To help control suction and evaporation of the water from the new mortar, the existing stones and mortar should be wetted to Saturated Surface Dry (SSD) prior to the placement on any new mortar.
5. The mortar should be applied by hand and in 1/4 inch lifts (especially in the last 1-1/2 inch to be repointed) as shown in Figure 16.27. Thicker lifts and use of a pressure gun (or grout gun as shown in Figure 16.28 and Figure 16.29) to apply the mortar (or grout) can be used behind the final 1-1/2 inch if required. All lifts should cure to thumbprint hardness prior to the application of the next layer as shown in Figure 16.30.

Immediately after repointing, the masonry units should be cleaned of all excess mortar. Application of a bond breaking agent, such as paraffin wax, to the face of the masonry units can aid in the clean-up process. (Figure 16.29 shows the wax on the face of the stones). It is recommended that use of metal tools or chemical cleaning agents be avoided unless tests to determine their effect on the masonry units have been performed and no adverse effect is noted.

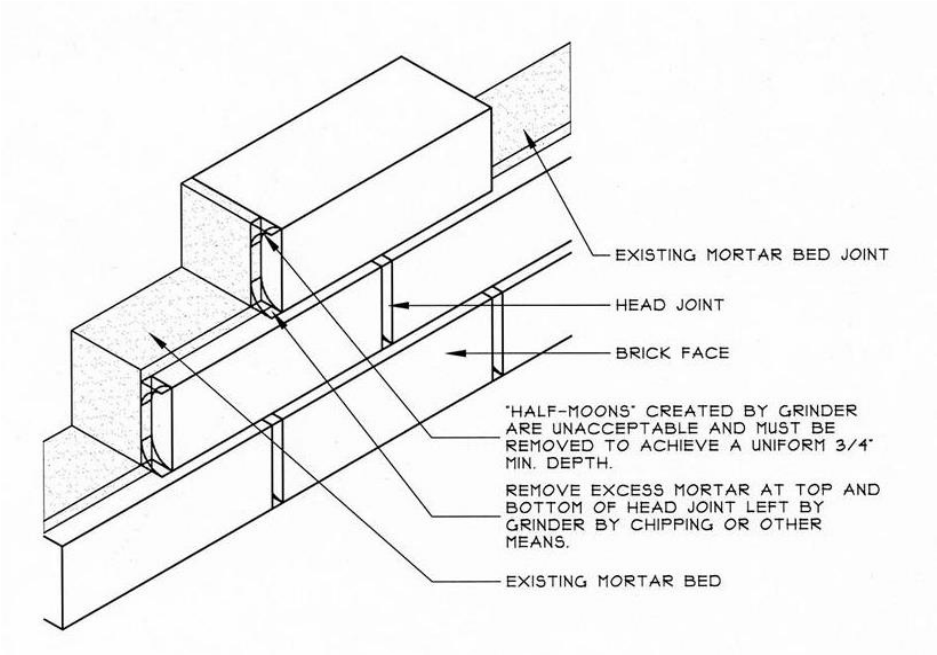
EXISTING MORTAR BED JOINT
HEAD JOINT
BRICK FACE
"HALF-MOONS" CREATED BY GRINDER ARE UNACCEPTABLE AND MUST BE REMOVED TO ACHIEVE A UNIFORM 3/4" MIN. DEPTH.
REMOVE EXCESS MORTAR AT TOP AND BOTTOM OF HEAD JOINT LEFT BY GRINDER BY CHIPPING OR OTHER MEANS.
EXISTING MORTAR BED

Figure 16.27 Preparation for Mortar Repointing

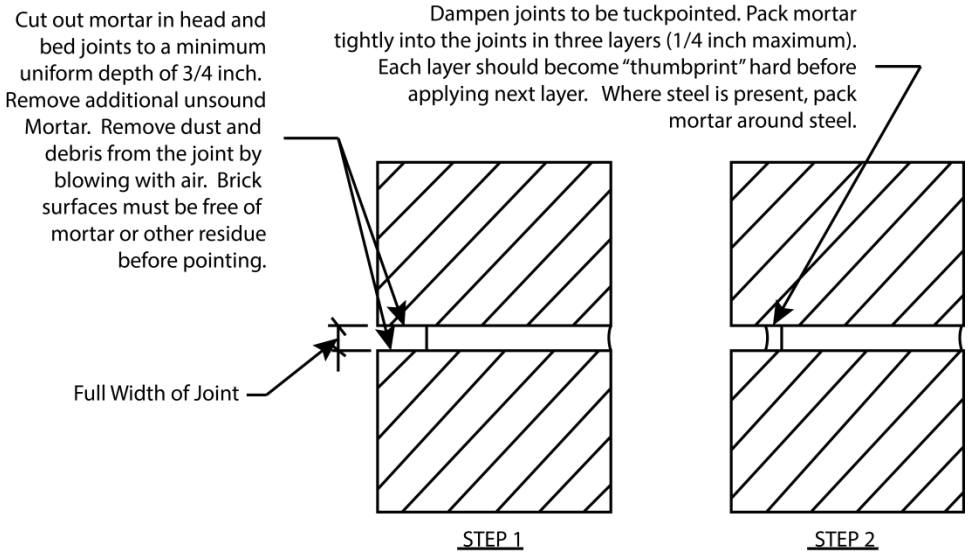
Cut out mortar in head and bed joints to a minimum uniform depth of 3/4 inch. Remove additional unsound Mortar. Remove dust and debris from the joint by blowing with air. Brick surfaces must be free of mortar or other residue before pointing.
Dampen joints to be tuckpointed. Pack mortar tightly into the joints in three layers (1/4 inch maximum). Each layer should become "thumbprint" hard before applying next layer. Where steel is present, pack mortar around steel.
Full Width of Joint
STEP 1
STEP 2

Figure 16.28 Mortar Repointing Schematic

Figure 16.29 Pressure Injection of Mortar (Note presence of paraffin wax on stones)

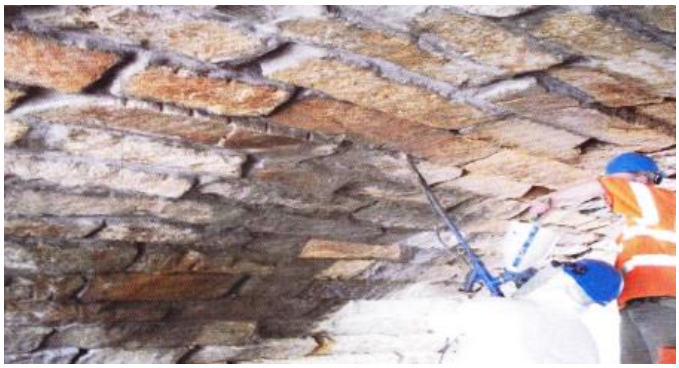

Figure 16.30 Pressure Injection of Mortar (Smaller nozzle size for neater application)

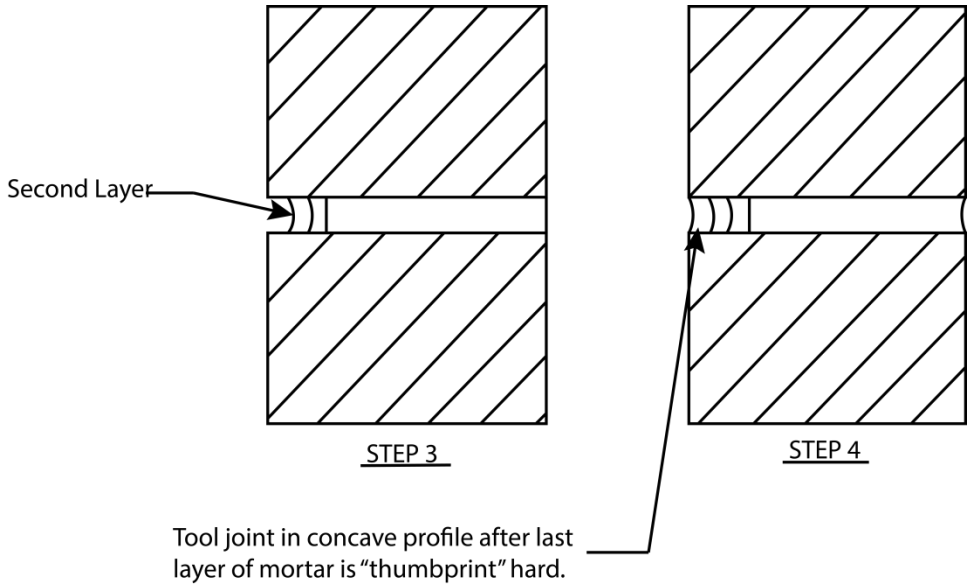

Figure 16.31 Mortar Repointing Schematic Showing Proper Geometry of Layers

16.3.2 Removal of Shotcrete Coating

The application of exterior coating, such as shotcrete, to masonry structures can trap moisture. The trapped moisture makes the structure more susceptible to frost damage. Frost can cause cracks in the coating (as shown in the example in Figure 16.32) and in the bridge mortar. The cracks then allow more moisture to enter, which then enlarges the crack during the freeze / thaw cycle. The coating can also trap moisture between the masonry units and the coating. If the coating does not allow the masonry units to dry out, spalling and delamination of the masonry units can occur. Additionally, this coating hides potential problems and signs of distress from the inspector, making visual inspection a much less effective tool.

It is recommended that gunite or shotcrete coating be removed when signs of distress are found. Wetness along the joint or efflorescence is often the first sign that the coating has water trapped behind it. Removal of the coating should be achieved through the use of hand tools or small pneumatic hammers and care should be taken to remove the coating without damaging the masonry units. Any damage that occurs to the masonry joints should be repaired by repointing.

Figure 16.32 Evidence of Degraded Shotcrete Coating. Macro View (left) and Magnified View of Inset on Left Showing Cracks and Efflorescence (right).

- Care should be taken when the coating is removed. The surrounding surfaces should be continuously monitored to ensure that the coating is not delaminating and potentially detaching, a dangerous situation which could cause a sudden collapse of a large section of the coating and injury to the workers. If any masonry units fall out during the removal of the coating, they should be replaced (see missing stone repair procedure outlined below).
- After the coating is removed, it is recommended that the surfaces be cleaned using a light abrasive blast cleaning (below 400 psi) to remove any remaining film and chisel marks. However, if damage to the masonry units is occurring, the process should be immediately stopped and another method of cleaning or a discontinuation of the cleaning should be performed. Low pressure water (preferably demineralized) should then be used to finish cleaning the surfaces. If repointing is required, follow the procedure in Section 16.3.1.

16.3.3 Repair of Spalling or Delaminated Masonry Units

Spalling occurs when the exterior face of the masonry unit begins to flake or chip off. Delamination occurs when the masonry unit separates into constituent layers. If the spalling or delamination is extensive, replacement of the whole area may be required. However, if the damage occurs over a small area and there is no evidence of water infiltration, repair can be accomplished.

The recommended repair is to use mortar to patch the face. The repair is typically completed for aesthetic purposes only. If the spalling is widespread enough to be a structural concern, in the opinion of an engineer, the delaminated stones will most likely require replacement. Please refer to section 16.3.4 for information on replacing stones. The mortar mix should be designed to match the color of the adjacent stones and finished to match the texture. The face of the masonry unit to be repaired should be cleaned with a wire brush, low pressure compressed air, or low pressure water prior to the mortar being applied. After the mortar has cured, the adjacent mortar joints should be repointed. At a minimum, mortar lines should be scribed to represent the joints.

16.3.4 Repair of Missing Masonry Units

This treatment is intended to replace a single stone or small group of stones. If larger areas of stone are missing or loose, a more extensive rehabilitation or restoration of the masonry is required. Missing stones can be indicative of a serious problem with the bridge, such as water infiltration. Therefore it is recommended that the underlying cause of the missing stones be discovered and addressed prior to replacing the stones.

If the original stones can be found, it is recommended to use the original stones in the replacement. If not, it is recommended to find stones that match the original stones. Clean the area where the stone will be replaced. Apply a bedding mortar (a layer of mortar that is applied below the masonry unit to be set) in the area of the stone to be replaced. Carefully reset the stones ensuring that they are well bedded. Take care to protect the adjacent masonry units. It may be required to shore the masonry unit in place until the bedding material and potentially the mortar used for repointing have cured. After the bedding mortar has cured, repoint the masonry joints following the repointing procedure provided in Section 16.3.1.

16.3.5 Repair of Slipped Masonry Units

It is recommended that loose slipped stones be removed and replaced (see missing stone procedure above). If the slipped stone is locked into its location and does not create a structure deficiency, it is recommended that it be left in place. The mortar adjacent to the slipped stone should be repointed (or pointed) to further help secure the slipped stone.

16.3.6 Repair of Cracked Masonry Units

Slab bridges that have transverse cracks cannot be repaired. Temporary shoring can be used until a permanent solution can be found, but the cracked slab requires replacement. Longitudinal cracks in slabs and cracks in arch barrels are not typically structural detriments and can be repaired.

- If there is water leaking through the crack, if must be addressed prior to repair of the crack (see water infiltration above). Only repair the crack after water no longer escapes

through it. Mortar injection can be used to repair the crack without having to excavate over the bridge.

- The joint must be cleaned using hand tools, low pressure compressed air, low pressure water or vacuuming. Mortar injection ports should then be affixed into or onto the crack. Make sure that there are ports attached at the ends of the crack to ensure complete filling. For cracks less than 1/4 inch wide, space the injection ports no further than 2 feet apart. For cracks over 1/4 inch wide, the ports can be spaced up to 3 feet apart.
- After the ports have been installed, the crack can be sealed using a stone patching grout. The injection should begin at the wider end of a horizontal crack or at the lowest point of a sloping or vertical crack. Adjacent stones should be monitored during the pressure injection to ensure there is no movement. If any movement is found, immediately stop the pressure injection.
- The grout should be placed in the first port and the filling should continue until it starts leaking out of the next port. The leaking port should then be temporarily capped and the filling should continue until maximum pressure is reached (5 psi, as shown on a pressure gauge). Only then should the pressure injection be moved to the next lowest port and the first port sealed. After the grout has cured, carefully remove the injection ports and fill the holes with grout or mortar.

16.3.7 Arch Deformation Repair

If the deformation of the arch is not detrimental to the structural capacity of the arch, as determined by an engineer, then the stones can be repointed to prevent further displacement of the masonry units. However, if the structural capacity of the arch does not meet the safety requirements for the structure, more extensive rehabilitation is required. (Presented in detail in an ensuing section)

The recommended repair for the displaced stones is the application of mortar or grout with a pressure gun to penetrate as far as possible into the joints. The last 1-1/2 inch of the joints should be repointed by hand (repointing procedure described in Section 16.3.1).

16.3.7.1 Relieving Slabs

One method of repairing an arch that had insufficient structural capacity is the installation of a reinforced concrete relieving slab, as shown in Figure 16.33. The purpose of the relieving slab is to help distribute the live load to the arch. The relieving slab distributes the live load over a wider area than either a directly applied point load (less than 2 feet of fill) or a pressure based on distribution through over 2 feet of fill. AASHTO LRFD Manual provides further information in the buried structure section, Chapter 12.

Figure 16.33 Reinforced Concrete Relieving Slab (Under Construction)

16.3.7.2 Moment Slabs

A common problem with masonry bridges is the lack of a crash tested barrier system and the difficulty inherent in attaching one. One possible solution is the use of a moment slab, as shown by the example in Figure 16.34. The moment slab can be placed over a masonry bridge if there is sufficient fill, as shown by the example in Figure 16.35, or it can be placed around an arch if the geometry allows.

Figure 16.34 Reinforced Concrete Moment Slab (Under Construction)

Figure 16.35 Moment Slab (indicated by arrow)

16.3.7.3 Repairing an Arch if Fill is Removed

Repairing an arch by removing the fill should only be undertaken if all other options have already been exhausted. A Professional Engineer should oversee be involved in the design and construction of this work. Prior to any excavation, the use of formwork should be investigated to determine its necessity. The barrel may appear to be stable when construction is first started; however, as the fill is removed over the arch, it is likely that the stones will shift. It is extremely important that the fill be removed from both sides of the arch in equal lifts as the unbalanced loading is detrimental to masonry arches. Shoring the arch can help to alleviate some of these concerns.

If construction vehicles will be traveling over the arch when fill is removed, or if the depth of fill in the final condition (after the work is complete) is going to vary from the existing condition, it is extremely critical that the arch be analyzed for the revised depths of fill. Masonry arches are very sensitive to variations in the depth of fill and even small changes can be detrimental to the structure.

One possible method of repair is to install a concrete saddle over the extrados of the arch. The concrete saddle is concrete that is poured over the arch to strengthen the arch and lock the masonry units into place. The concrete saddle can either be used over a deteriorated portion of the arch or over the full structure. However, since it is necessary to remove the soil over the arch for the installation of a concrete saddle, the preferred method of strengthening a solid structure that is structurally deficient for current loading is to install a relieving slab to help distribute the load (refer to Section 16.3.7.1). The purpose of the saddle is to reinforce the area where the saddle is applied, as well as to help distribute the load around the damaged area.

Excavation over the arch can also be required for the installation of a relieving slab or to remedy drainage and water infiltration problems. In any case where fill is removed, it is recommended that well graded fill be used to replace the existing material. Use of a geotextile material for separation is also recommended.

16.3.8 Scour Protection

Substructure failure due to channel scour can cause catastrophic collapse of a masonry bridge. The substructure of a masonry bridge is typically comprised of unconnected masonry units. The small units are much less likely to be able to span over a scour or undermining hole than a reinforced concrete footing. Masonry bridges are therefore much more susceptible to scour damage. Once the stones begin to settle into the undermined area, continuity is lost and an accelerated loss of stones is likely to occur. If unchecked, collapse of the arch barrel is likely.

Scour holes should be filled with large granular material (6 inch to 8 inch minimum stone size). The actual required stone size should be sized by a hydraulic engineer. Higher velocity streams require larger stone sizes. If undermining of the substructure has occurred, it must be repaired by placement of concrete under the undermined area or another acceptable repair.

If undermining of the arch has occurred, there are several methods that can be used to fill the voids.

- Grout bags can be placed in the voids
- Tremie placed concrete can placed in the voids
- Pressure injected grout can be used to fill the voids

- Construct cofferdam, dewater, and pour subfooting

Typically a combination of the above methods will be required by the engineer. Dewatering is required. Refer to Chapter 4 and Section 13.4.8 for further guidance on scour and undermining repair.

When to Call the Engineer

Call the Engineer for guidance on undermining repairs.

16.3.9 Repairing Masonry Veneers

Masonry veneer failure is typically more of an aesthetic concern than a structural concern. However, masonry falling out of the veneer can be a safety concern. The masonry veneer should be sounded to determine the extent of the delamination. Anchors can be installed to re-attach the veneer to its supporting wall if the veneer is delaminated, but has not yet fallen. The sketches on the following pages in Figure 16.36 through Figure 16.38 illustrate several methods to re-attach the veneer. Figure 16.37 shows an expansion anchor suitable for use in a solid backing substrate.

If a masonry veneer has fallen away from its supporting wall, then re-construction will be necessary. Prior to starting any reconstruction, it should be determined if the area surrounding the failed area is structurally sufficient. Soundings can be taken around the area requiring repair to determine the integrity of the surrounding veneer. The surrounding area should then be shored as required. After the shoring has been completed, the veneer can be reconstructed. The use of anchors is highly recommended in the replacement of the veneer. Please refer to section 16.3.4 above for additional direction on the replacement of the missing masonry units.

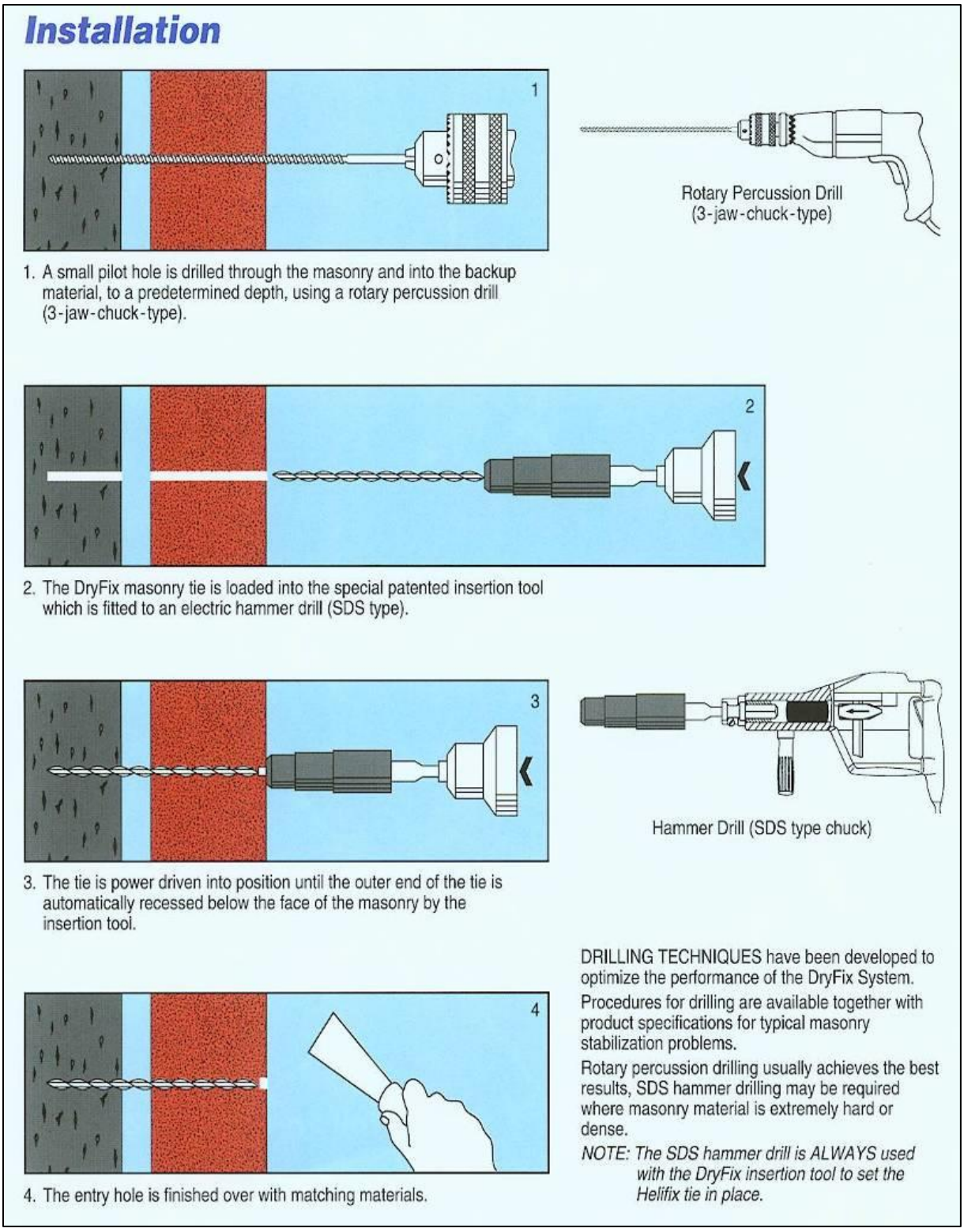
Installation
1
Rotary Percussion Drill
(3-jaw-chuck-type)
1. A small pilot hole is drilled through the masonry and into the backup material, to a predetermined depth, using a rotary percussion drill (3-jaw-chuck-type).
2
2. The DryFix masonry tie is loaded into the special patented insertion tool which is fitted to an electric hammer drill (SDS type).
3
Hammer Drill (SDS type chuck)
3. The tie is power driven into position until the outer end of the tie is automatically recessed below the face of the masonry by the insertion tool.
DRILLING TECHNIQUES have been developed to optimize the performance of the DryFix System.
Procedures for drilling are available together with product specifications for typical masonry stabilization problems.
Rotary percussion drilling usually achieves the best results, SDS hammer drilling may be required where masonry material is extremely hard or dense.
NOTE: The SDS hammer drill is ALWAYS used with the DryFix insertion tool to set the Helifix tie in place.
4
4. The entry hole is finished over with matching materials.

Figure 16.36 Installation of Helical Anchor

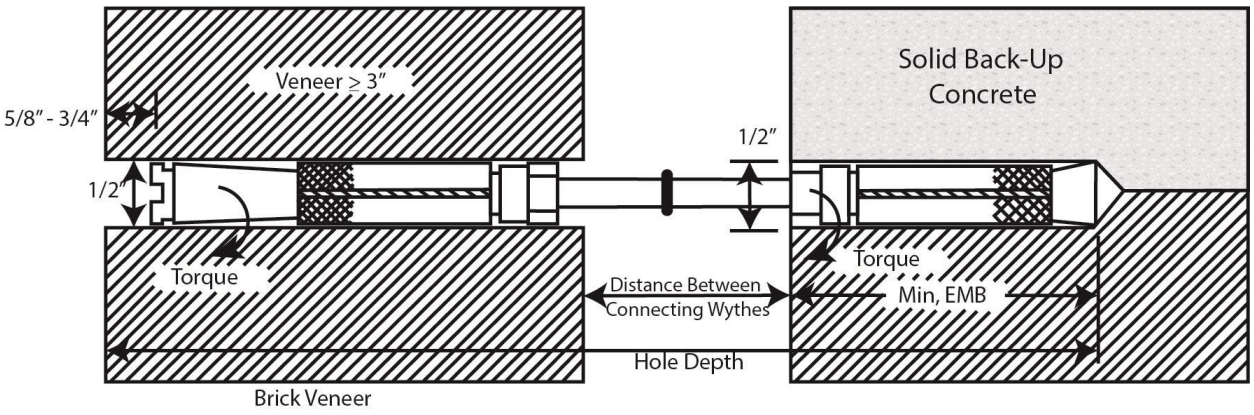

Figure 16.37 Expansion Anchor for Masonry Veneer Repair

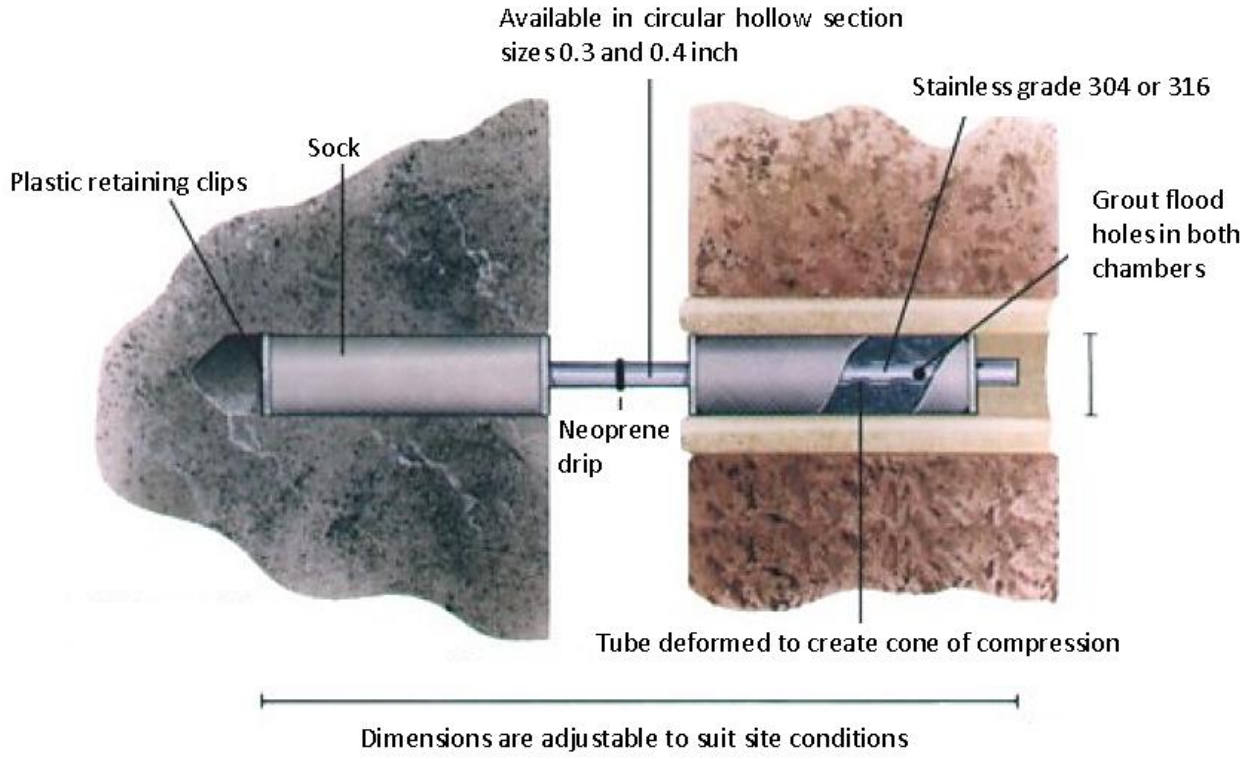

Figure 16.38 Masonry Veneer Repair with Adhesive Anchor

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Chapter 17 - Movable Bridges

A movable bridge is a bridge across a navigable waterway, or other travel way, that has at least one span which can be temporarily moved to increase the vertical clearance for objects passing underneath. An example of a bascule bridge is shown in Figure 17.1. Bridges with a movable span are a design option when it is not feasible to obtain the necessary vertical clearance for objects passing beneath the structure.

Figure 17.1 Bicentennial Bascule Bridge over the St. Joseph River

Each movable bridge has specialized features and unique mechanisms for movement. The following sections cover common types of movable bridges, including information on the operation and maintenance of the structure and the movement mechanisms. The maintenance of these specialized bridges requires a diverse team collectively capable of operating the structure as well maintaining the mechanical, electrical, pneumatic or other movement mechanisms. The duties, skills, and responsibilities of the bridge maintenance crews are defined by the bridge owner and are not uniform. Some crews are capable of performing maintenance on these structures, while other owners may elect to contract the work.

Movable bridges are unique structures as they integrate conventional structural components with mechanical systems and electrical power and control systems. These structures are also different from most highway bridges in that they actively facilitate the flow of both vehicular and waterborne (typically) traffic.

Movable bridges may be located in marine environments, snow and ice zones, and carry heavy traffic loads. Section loss due to corrosion in the structural components of the bridge can lead to misalignment, bending, or deformation in lock switches, open gears, and the electrical systems found on movable bridges. Heavy loads and thermal expansion effects can cause misalignment of the structures and lead to malfunctions in the operation of the bridge.

Maintenance and performance monitoring of movable bridges is often more critical than for fixed bridges given the dual service role and the potential for deterioration and other problems with the integrated systems that are essential for ensuring their operation and safety.

Recommendation

Movable bridge maintenance workers need to be constantly monitoring and inspecting structural, mechanical, and electrical elements.

17.1 Common Types of Movable Bridges

Three categories of movable bridges comprise over 95 percent of the total number of movable bridges within the United States. There are three basic types of movable bridges; (1) bascule, (2) lift, and (3) swing. A fourth, less common type, is a retractile bridge. Each type of movable bridge has variations in the design and machinery for opening and closing of the structure. For instance, bascule bridges can be further subdivided as rolling type (also call a Scherzer), a simple trunnion type (commonly referred to the Chicago style), and a modification of the trunnion (referred to as a Strauss). Each type of movable bridge has their own unique problems associated with each depending on their particular opening mechanism. Types of Movable Bridges and their associated displacement are shown in Table 17.1:

Table 17.1 Movable Bridge Types and Corresponding Displacement

Movable Bridge Type	Displacement Type
Bascule (Trunnion)	Rotation about a fixed horizontal axis
Rolling Lift Bascule (Rolling Lift)	Rotation about a fixed horizontal axis that simultaneously translates
Swing	Rotation about a fixed vertical axis
Retractile	Translation along a fixed horizontal axis
Vertical Lift	Translation along a fixed vertical axis

17.1.1 Terminology

Terms commonly used regarding movable bridges are defined below.

Auxiliary Counterweight: Secondary weights that offset the shifting weight of main counterweight ropes as they travel from one side of the sheave to the other during operation.

Bascule Pier: A pier designed to house the machinery and counterweight of a bascule bridge.

Counterweights: Weights fixed to the bascule girders to minimize the force required to move the leaf.

End Locks (also called *tail locks* or *heel locks*): Locks which prevent uplift at the long end of a bascule leaf by preventing the short end from deflecting under live loads. End locks are retracted prior to the start of the opening of the bridge.

Flanking Span Trusses: The trusses at either end of a lift span truss; they do not move.

Leaf: The movable span of a bascule bridge.

Lift Span Truss: The structural span designed to move up and down, parallel to the roadway.

Live Load Bearing: Support blocks that the span rests on while in the closed position for trunnion bascule, rolling lift bascule, and vertical lift bridges

Lockbar: A bar which engages a socket to prevent span movement in either the open or closed position.

Lockbar Actuator: A control system for a motor to the engage or retract a lock bar.

Lockbar Socket: The female end that typically contains bushings to guide the lockbar into a locked position.

Pinion: Any toothed gear of small size as compared with the gear which it engages. The pinion is usually the driver.

Pivot Pier: The pier on which the center bearing swing bridge rotates

Rack: A gear with teeth spaced along a straight line and suitable for straight-line motion.

Rest Pier: Piers designed to carry the load of the swing span of a swing span bridge in the closed position.

Segmental Girder: The arc shape end of rolling lift bascule bridge leaf.

Sheave: On vertical-lift bridges, a large diameter pulley with annular grooves, over which the counterweight ropes pass.

Span Brakes: Brakes which hold the movable span and stop it during emergency conditions.

Span Guides: Guides which are used to restrict the lateral and longitudinal movement of the span during operation.

Span Lock: The lock used to transfer vehicular live load between the leaves of a double-leaf bascule bridge and to ensure the span is in the closed position for single-leaf bascule and vertical lift bridges.

Stabilizing Machinery: Components that support the span when it is in motion.

Toe and Heel: Terms used to represent the long and short ends of a movable bridge. For example, a bascule bridge lifts up and rotates around what is called a trunnion. The long end (toe) usually spans over the water and the short end (heel) connects to the abutment. A counterweight makes up for the weight difference between the toe and heel.

Track Segment: A part or unit of a circular track used to carry the rollers of a rim bearing draw-span.

Track Girder: A beam or girder carrying a track.

Trunnion Bearing: A bearing which allows rotation of the trunnion shaft and supports the weight of the span when in a raised position.

Trunnion Shaft: A short axle that allows the rotation of bascule leaf typically about a common center line and mounted in the trunnion bearings.

17.1.2 Bascule Bridges

Bascule bridges are the most common movable bridges in operation today. They originated with medieval draw bridges with no counterweights. They have evolved to include

counterweights, which has permitted increased size and utility. The counterbalances on modern bascules act to balance the weight of the leaf about its support.

Bascule bridges can be constructed with one or two leaves spanning the channel. A single-leaf bascule is supported by an abutment or pier and the single leaf spans the entire navigation channel. Only a single set of span drive and stabilizing machinery is required. However, the machinery may be significantly larger than that found on a double-leaf bascule spanning a channel of similar width. A schematic example of a single leaf simple trunnion bascule bridge is shown in Figure 17.2.

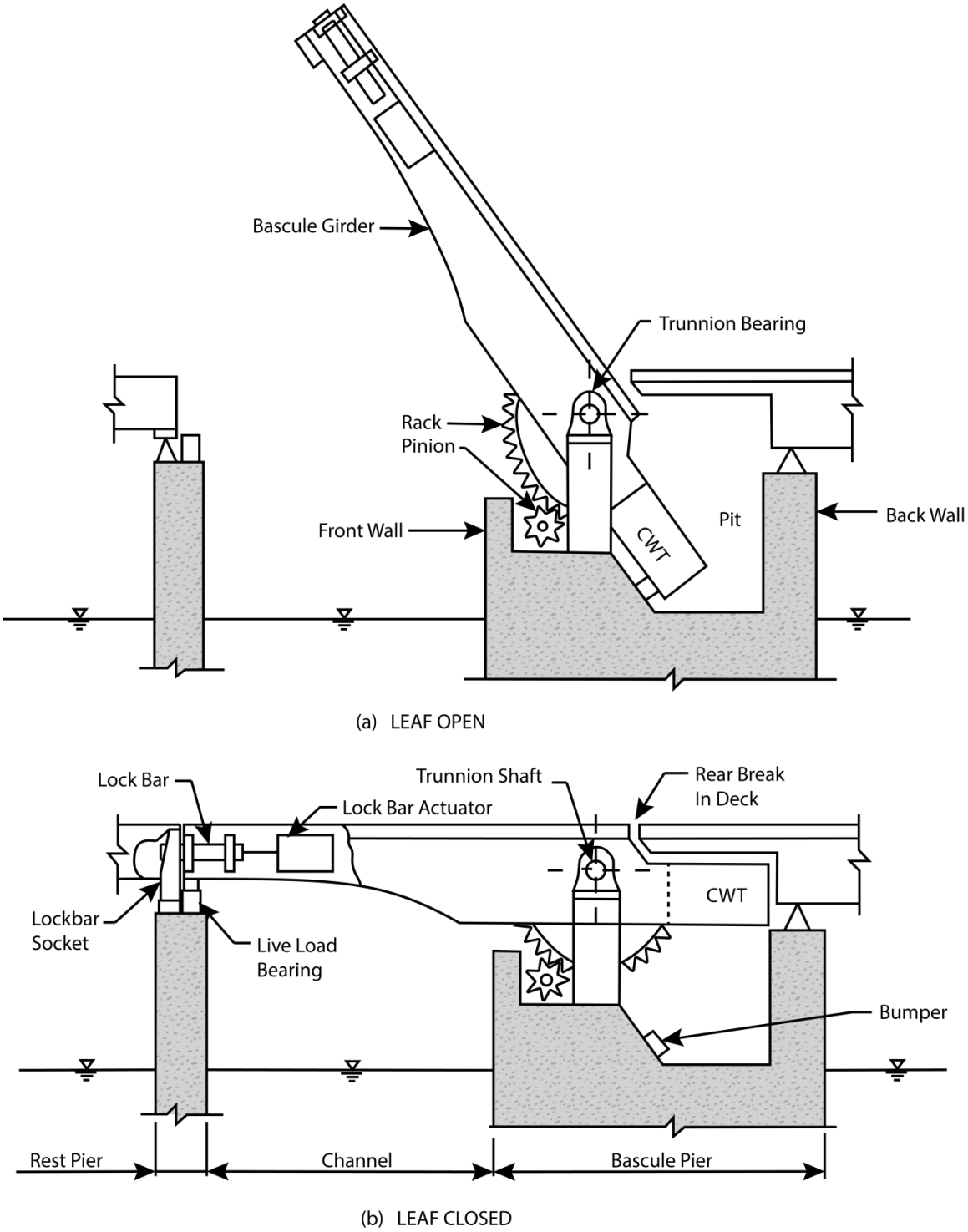


Figure 17.2 Single Leaf Simple Trunnion Bascule

A bridge with two leaves is called a double-leaf bascule, an example of which is shown in Figure 17.3. The two leaves usually meet at the center of the navigation channel. In double leaf bascule bridges, span locks are used to connect the tip ends of two cantilever bascule leaves; therefore, both leaves are forced to deflect equally. This ensures compatibility of the deck deflections from the opposing span leaves under traffic loads. The span locks typically consist of two main components: the receiver (lock bar socket) and the rectangular lock bar. The leaves

would only need to be one-half the length of a single-leaf bascule leaf to span the same channel. However, double-leaf bascules require span drives and stabilizing machinery for both leaves, as the two leaves are independent of each other and, therefore, require independent machinery systems for operation.

The raised leaf of a bascule bridge opens the waterway for the passage of vessels and also provides a barrier to roadway traffic the moment it begins to open and remains a barrier throughout the cycle. An example is shown in Figure 17.3.

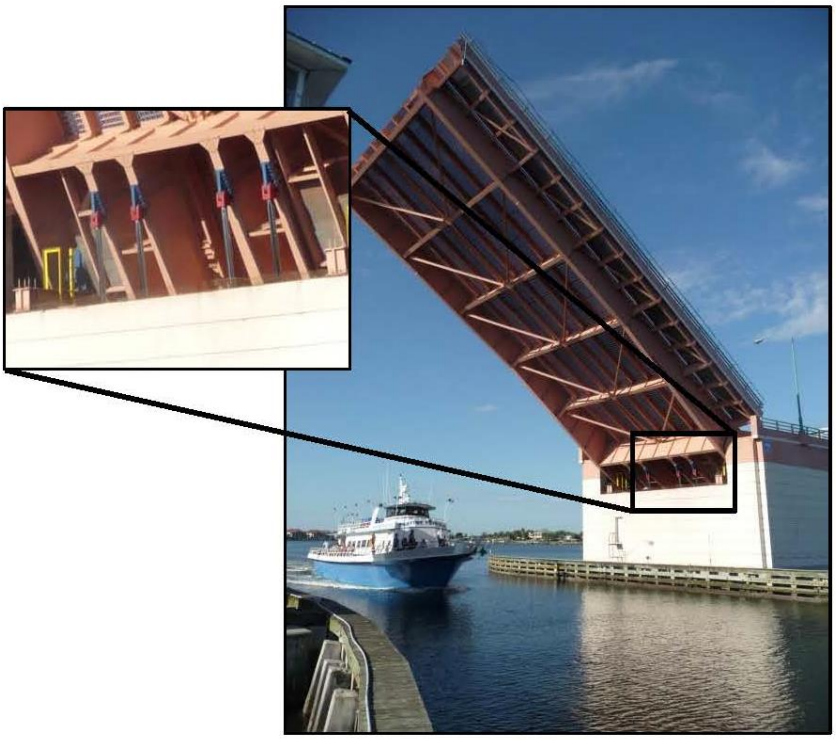


Figure 17.3 Cylinder Driven Simple Trunnion Bascule

A variety of bascule bridges exist, but the two most common types are the rolling bascule and the trunnion type bascule. See Figure 17.4 left for a simple trunnion bascule. The rolling bascule utilizes tread and track plates and the straight rack (shown in Figure 17.4 right). The trunnion type utilizes a curved segmental rack which is bolted or fabricated to a structural steel support and connected to the bascule girder.

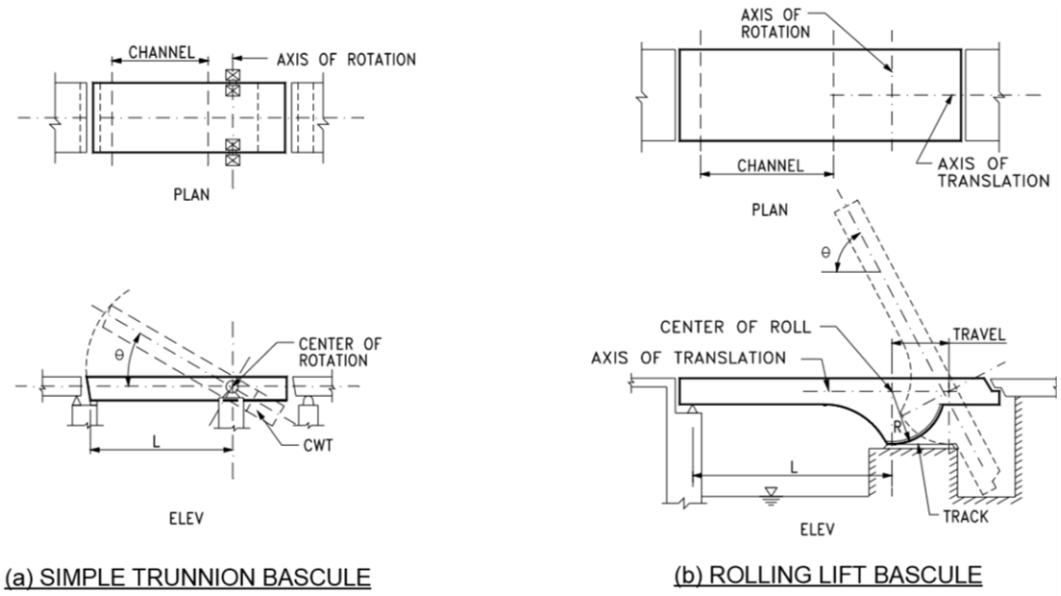


Figure 17.4 Types of Bascule Bridges (a) Simple Trunnion (b) Rolling Lift Bascule (Courtesy of Wisconsin DOT)

The pinion is the smallest gear in a gear set. The pinion is driven by the span drive machinery and engages the rack to move the span (see Figure 17.5). Simple trunnion bascules can also be operated by hydraulic cylinders. The cylinders are typically mounted on the bascule piers and the piston rods are connected to the leaves in a manner such that extending or retracting the rods rotates the leaf. See Figure 17.6 for an example of a hydraulic cylinder on a trunnion bascule bridge.

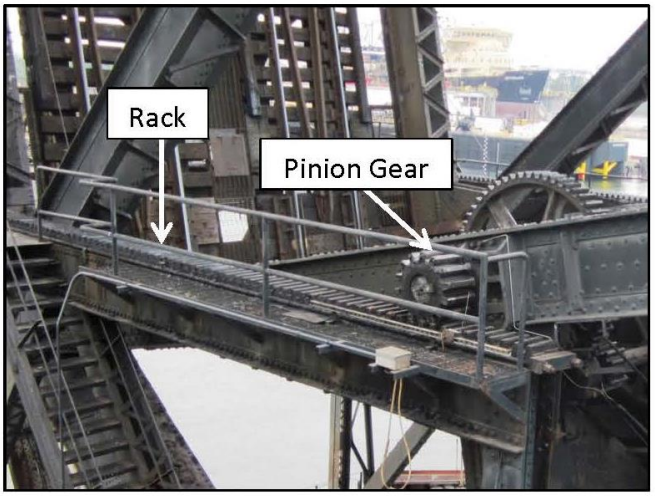


Figure 17.5 Rack and Pinion Gear

Figure 17.6 Hydraulic Cylinder on Trunnion Bascule Bridge

Counterweights are typically fixed to the bascule girders to minimize the force required to move the leaf. To achieve this, the trunnion axes are located at or near the center of gravity of the leaf including the counterweight.

For most bascule bridges, the ideal balance condition is slight toe, or span, heaviness, and it is the condition for which the bridge and mechanical components were designed. Operation during unbalanced conditions will place excessive wear on the bridge's structural and mechanical components, leading to costly repairs or possible safety concerns. During opening and closing, even a perfectly tuned and maintained leaf is expected to exhibit some friction, however, disruption to the alignment of leaves or any part of the mechanical system increases the friction. A balance test can be performed to determine the balance of the leaf and the friction present in the mechanical system. More details on balance testing are provided in Section 17.3.21.2.

Rolling lift bridges (also called Scherzer type) are characterized by a curved end section (or a "fish belly") of the girders or trusses. During bridge operation, the machinery creates a rotation about an axis that translates to movement in the horizontal direction. As the tread rolls along the tracks, the bascule leaf rotates open or closed.

Figure 17.7 depicts a single-leaf deck type Scherzer bascule. As the leaf tilts open to clear the channel the curved ends of the girders roll away from the channel. Slippage between the segmental girder treads and the tracks on which they ride is prevented by lugs or teeth that mechanically engage sockets. The protruding lugs are typically located on the track and the receiving holes or notches are in the segmental girder treads. Figure 17.7 shows the rack is located below the pinion, which is common on older Scherzer-type bridges. Some newer bridges have the rack located above the pinion.

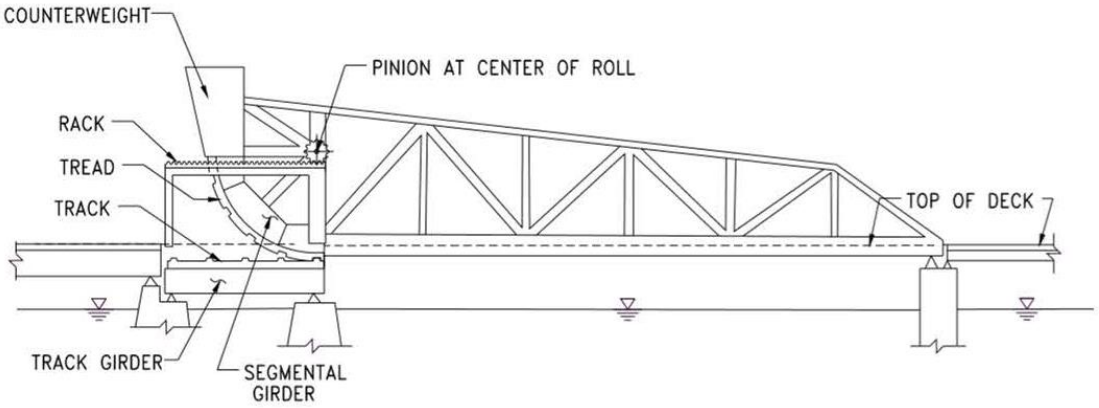


Figure 17.7 Through Single-Leaf Scherzer Rolling Lift Bascule

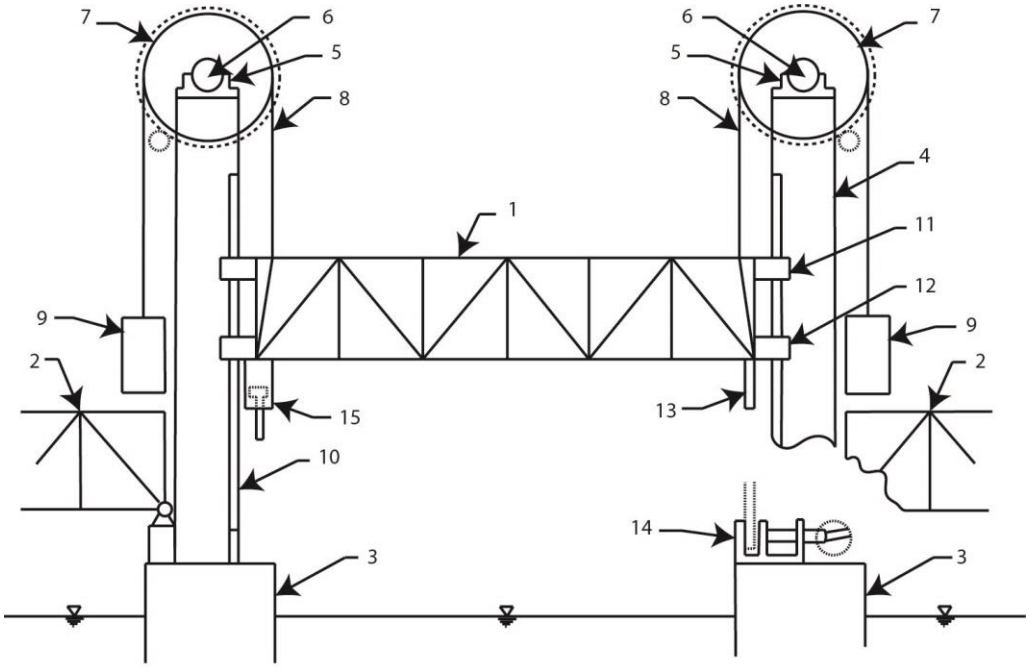
17.1.3 Vertical Lift Bridges

The spans on vertical lift bridges are lifted up and down just like an elevator. These bridges have a tower on each end, each of which encloses counterweights that offset the weight of the lift span. There are two main types of vertical lift bridges: Span Driven vertical lift bridges and Tower Driven vertical lift bridges. On Span Driven vertical lift bridges, the operating machinery is located on the lift span. Operating ropes, which are connected to the ends of the lift span and the counterweights, are draped over sheaves located at the top of the towers. An example of a Span Driven vertical lift bridge is shown in Figure 17.8.

Figure 17.8 Span Driven Vertical Lift Bridge

For Tower Driven vertical lift bridges, the operating machinery is located at the tops of each tower, with the lift span and counterweights located on either side of the operating drums. An example of a tower driven vertical lift bridge is shown in Figure 17.9. A schematic of a vertical lift bridge and its components is presented in Figure 17.10. Examples of vertical lift bridges include the Roosevelt Island Bridge between Queens, New York, and Roosevelt Island in the East River, and the Commodore Schuyler Heim Bridge at the Port of Los Angeles. The longest vertical lift bridge in service in the world is the Arthur Kill lift bridge between Staten Island, NY and Elizabeth, NJ with a span of 559 feet.

Figure 17.9 Tower Driven Vertical Lift Bridge



1. Lift Span Truss
2. Flanking Span Truss
3. Pier
4. Tower
5. CWT Trunion Bearing
6. CWT Trunion
7. CWT Sheave
8. Main CWT Ropes
9. Main CWT
10. Span Guide
11. Upper Span Guide
12. Lower Span Guide
13. Centering Bar & Span Lock Socket
14. Span Lock Actuator & Lockbar (Retracted)
15. Air Buffer

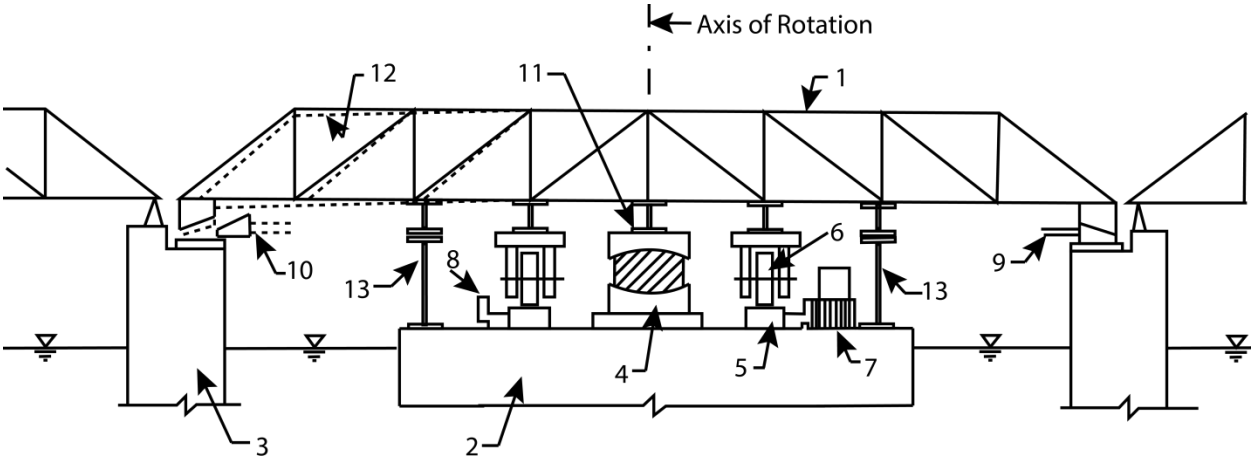
Figure 17.10 Vertical Lift Bridge Schematic

17.1.4 Swing Bridges

Swing bridges have an axis at their center and move by spinning horizontally until the bridge is perpendicular to its previous position, therefore creating two channels through which boats can run, as shown in the photographic example in Figure 17.11.

Figure 17.11 Center Bearing Swing Span Bridge

Swing bridges are also occasionally cantilevered out from one side of the channel when there is not space for the axis to be located right at the bridge's center. Swing bridges are rarely built today, due to slow operation of the movable span and the space needed for movement. A schematic of a center-bearing swing bridge is shown in Figure 17.12.



1. Swing Span (Draw)
2. Pivot Pier
3. Rest Pier
4. Center Bearing
5. Track
6. Balance Wheel
7. Pinion
8. Rack
9. End Wedges (Extended)
10. End Wedges (Withdrawn)
11. Distribution Framing
12. Deflected Position (Wedges Withdrawn)
13. Live Load Wedges

Figure 17.12 Center-Bearing Swing Bridge Schematic

The entire main span weight of a center pivot swing bridge rotates about and is supported by the center pivot bearing. Bronze disc type or an anti-friction type spherical thrust bearings may be used for center pivot bearings. Balance wheels help stabilize the span from any lateral forces on the span while the span is in motion. The wheels are set to a small clearance (approximately 0.1 inch) with tapered track, and only make light contact due to imbalance and/or wind loads.

A rim bearing swing span has an upper and lower track with large tapered rollers that rotate between the tracks. This tracks and rollers support the entire weight of the span during operation. Radial tension rods connected to a central hub hold the tapered rollers in alignment as the span rotates. This hub rotates about a vertical pin mounted to the pier. The rollers should rotate with no sliding or abnormal noises.

The ends of a swing bridge deflect during operation, and must be raised to meet the approach roadway level. This acts to stabilize the span under vehicular traffic. Several types of end lift machinery are used including: end wedges, end toggles, and eccentric cams. Most systems utilize crank mechanisms to lock the devices in place. The mechanisms are usually driven by a system of gears and a motor. During operation of a swing bridge, the ends of the swing span will deflect downward. In order to provide a smooth transition to the approach spans, and stabilize the bridge under load, end lift mechanisms are used to secure the swing span into place.

17.2 Operation of Movable Bridges

When a movable bridge is to be opened, a series of steps take place. The traffic lights turn to red, the alarm bell rings, and the gates are lowered. Next, the bridge lock is released allowing the bridge to move. When the span is fully opened, sensors automatically stop the machinery. The operator can override the system at any time during the movement.

While all movable bridges follow the same general procedure for operation, each bridge will have unique operating procedures which will be detailed in the Operations and Maintenance Manual for the bridge. To properly maintain the movable bridge, a maintenance worker must familiarize himself with the Operations and Maintenance Manual of the bridge, as well as the design plans for the structure (if available). Figure 17.13 provides an example of a table of contents for a movable bridge maintenance manual.

INDEX

OPERATION, MAINTENANCE, AND LUBRICATION MANUAL

JOB NO.: 10314-G

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Reducers	Section I
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Lubrication Data	Section IV
Steward Machine Co., Inc.'s Shop Drawings	Section V
Pintsch Bubenzer Brakes	Section VI
Spare Parts List with Seals and Gaskets	Section VII
Suppliers/Vendors	Section VIII
Warranties	Section IX

Figure 17.13 Example of a Table of Contents of a Bridge Specific Movable Bridge

Live load bearings are found on trunnion bascule, rolling lift bascule, vertical lift bridges and swing spans. Live load bearings are support blocks on which the span rests while in the closed position. On bascule bridges, the live load bearings are located forward of the trunnion and hold the main girder up. Live load bearing assemblies transfer vehicular live load from the movable span to the adjacent spans and piers when the bridge is in the closed position. An example of a live load bearing assembly is shown in Figure 17.14. The closed position is shown in Figure 17.14 (left), and the shoe (bearing) is shown in the open position in Figure 17.14 (right). Live load bearings are typically located at the four corners of a lift span, and in front of or behind the center of rotation for bascule bridges. The wedges located on the enter pivot pier of a swing span also serve as liveload bearings. Load bearing assemblies are comprised of an upper casting with a rounded surface (mounted to the span) and a flat strike plate mounted to the pier (or other fixed structure). The bearing and the strike plate are attached to the span and pier respectively, using bolts (and shims if necessary for adjustment purposes).

Figure 17.14 Live Load Wedges and Shoes
Closed Position with Wedge (left). Live Load Shoe (right).

Movable bridge machinery may be classified as either span drive or span stabilizing machinery. Span drive machinery are the components that move the span. Stabilizing machinery supports and restrains the span when it is at rest and when in motion.

Span drive machinery is similar across movable bridge types because the machinery converts the low-torque rotation of a motor (electric or internal combustion) to the high torque force required to move the span. In electro-mechanical drives, for instance, gearing is used to convert the low-torque, high-speed rotation of the motor to high-torque, low-speed rotation for moving the span. Various combinations of electrical, mechanical, and hydraulic components have been assembled to form span drives. An example of a span drive and its components is shown in Figure 17.15.

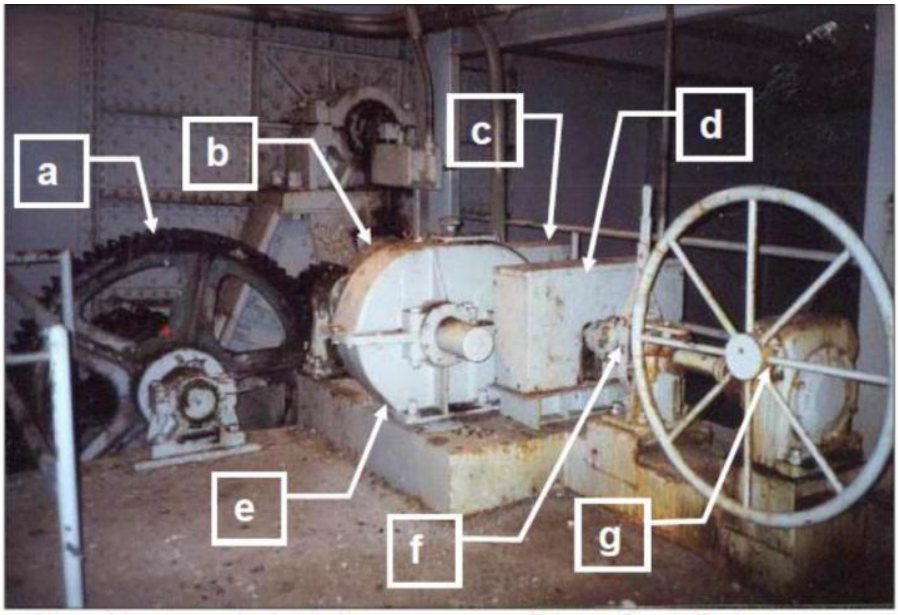


Figure 17.15 Components of a Span Drive

Stabilizing machinery comprises electro-mechanical and/or hydraulic components that support the span when it is in motion and when it is at rest. The machinery components and assemblies

are usually mechanical but fluid power (air and liquid) components are also utilized. General examples of stabilizing components for bascule bridges are the trunnion shafts and trunnion bearings (see Figure 17.16), span locks, centering devices, and buffers. On vertical lift bridges, stabilizing machinery includes the counterweight sheaves with trunnions and bearings, the main counterweight and auxiliary counterweight ropes, centering devices, span locks and guides, and buffers. Swing bridges are stabilized using center pivot bearings and balance wheels, rim girders, rollers and spider assemblies, center wedges; end lifts; centering latches or centering mechanisms, buffers, bumpers, and rigid stops.

Figure 17.16 Trunnion Bearing

The span motor and motor controller are used to slow the span during operation. Span brakes are not typically utilized for that function, but are used for holding the movable span and stopping it during emergency conditions.

The electrical, mechanical, and structural components of a movable bridge function as a system, which means that failures or deficiencies in one component will probably result in other components being affected. This means that a deficiency found at one item would most likely affect other elements.

17.2.1 Counterweights

Large blocks of concrete, called counterweights, are used to off-set the weight of the leaves on bascule spans or the lift spans on vertical lift spans. The weight of counterweight is designed to "balance" the system so the size of the mechanical systems and motors required to lift and close the leaf is minimized. A movable span operating in an unbalanced condition may cause overstress or failure of the mechanical or structural elements.

For lift bridges, auxiliary counterweight are secondary weights that are used to offset the shifting weight of main counterweight ropes as they travel from one side of the sheave to the opposite side during operation. The counterweights are made up of concrete and steel. The wire ropes holding the counterweight are guided by rails, sheaves, and termination points. Balance chains are used to help maintain span balance as they offset the weight of the wire rope as it spools during operation. An example of a counterweight assembly with auxiliary counterweights attached is shown in Figure 17.17.

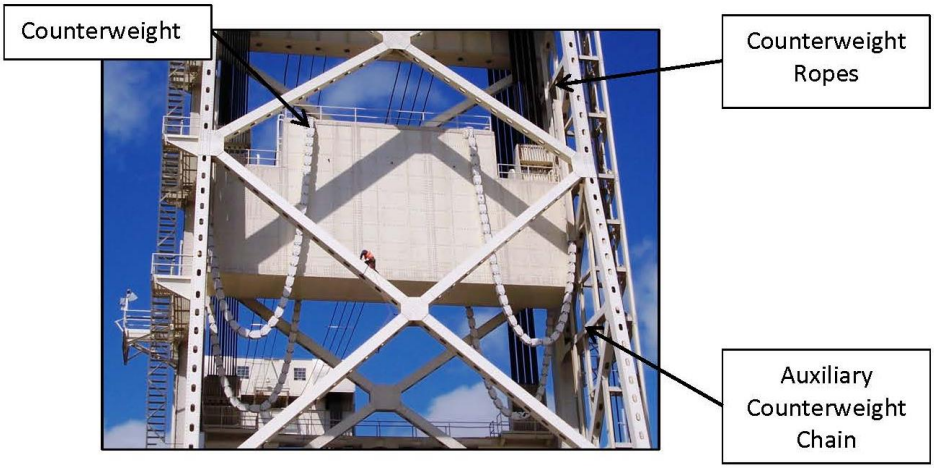


Figure 17.17 Counterweight Assembly

Throughout the lifetime of the bridge, counterweight adjustments are necessary. Short term adjustments compensate for ice or snow accumulations, for example. Adjustments to balance leaf weight changes due to activities such as repaving or painting are commonly performed. When the 250-foot long High Street Bridge in Alameda County, California was refurbished in 1996, 25,000 pounds of paint and primer were removed from its two bascule leaves. The counterweights had to be adjusted before and after repainting the span.

When to Call the Engineer

Whenever work is being performed on the bridge that will add or remove weight to either the movable span or the counter weight, the Engineer should be consulted to determine what steps, if any, will be required to keep the bridge in balance.

A dramatic example of the need to maintain proper counterbalance was shown by an accident on Chicago's Michigan Avenue Bridge on September 20, 1992. The two-level, double-leaf Bascule Bridge was undergoing repairs, and the concrete paving had been stripped off both the upper and lower decks. A large crane was parked behind the trunnion of one leaf, just above a counterweight that had not been lightened to compensate for the paving removal. Safety (span) locks may also have been improperly engaged or defective. The opposite side of the bridge was opened to allow a boat to pass. When it closed and mated with the side that had remained down, the static half was jarred enough to release its unbalanced energy. According to the Chicago Tribune, the leaf "shot up in the air". According to an analysis in the Journal of the American Society of Mechanical Engineers, "The rapid rotation of the bridge ripped it from its trunnion bearings and the entire span slammed to the bottom of the counterweight pit." Six people were injured as they scrambled out of a bus struck by flying debris, and the rear window of an occupied car was smashed by the wrecking ball attached to the crane as it fell from the bridge.

17.2.2 Mechanical Systems

Most movable bridges are powered by either electric-mechanical or hydraulic-mechanical drives with power driven pinions operating against racks or by hydraulic cylinders. A schematic of typical mechanical components is shown in Figure 17.18. A small number are hand powered for normal operation. Some smaller bridges use hand power for standby operation. An example of a brake and hydraulic span drive system is shown in Figure 17.19.

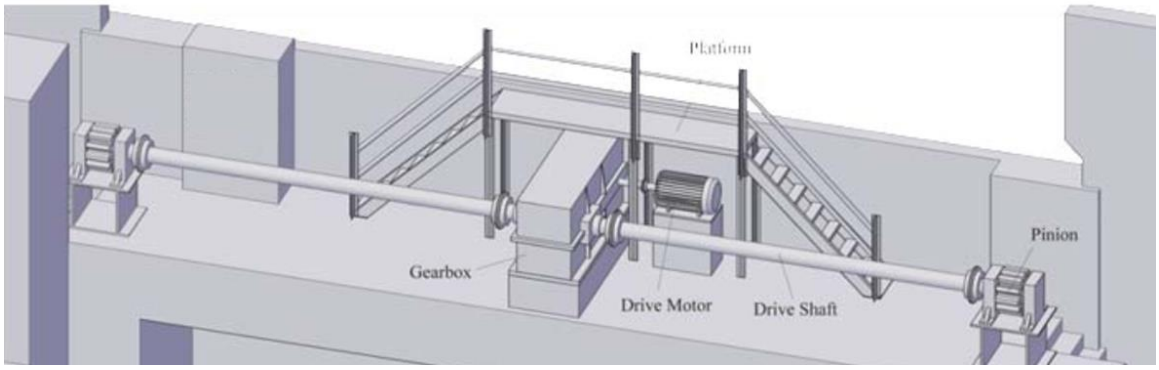


Figure 17.18 Mechanical/Electrical System of Representative Movable Bridge

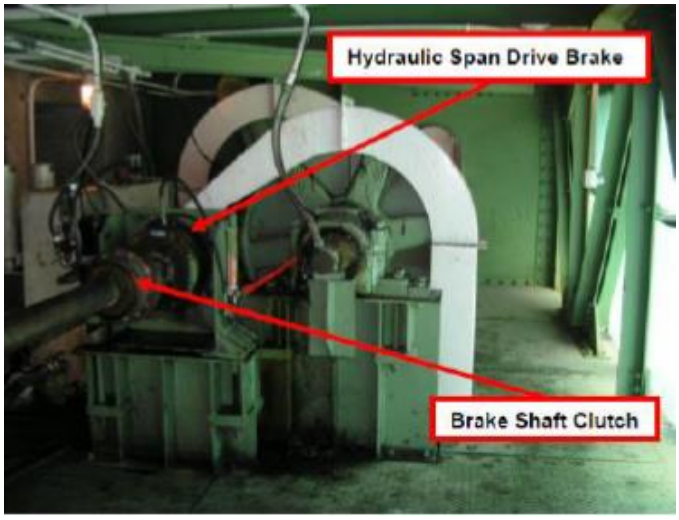


Figure 17.19 Brake and Hydraulic Span Drive System

The following components are often found on various types of movable bridges:

- Open Gearing: Open gearing transmits power from one shaft to another and alters the speed and torque output of the machinery. An example of open gearing is shown in Figure 17.20. Beveled gears change the direction of rotation.

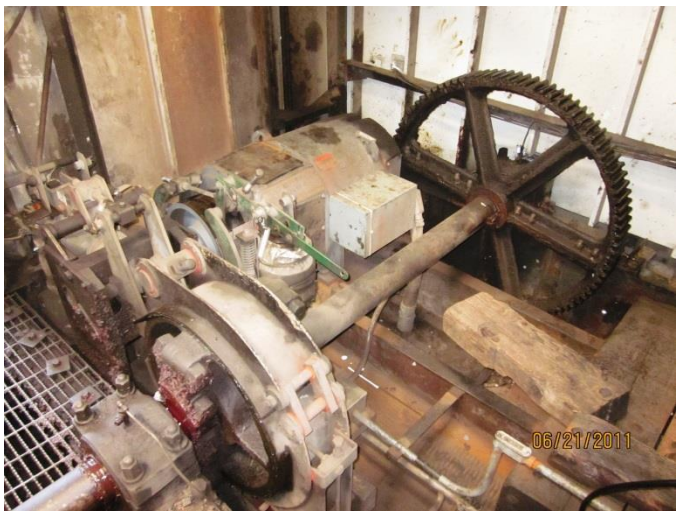


Figure 17.20 Open Gearing Operating Machinery

- Speed Reducers Including Differentials (Closed Gearing): Speed reducers including differentials serve the same function as open gearing. Differentials and speed reducers are typically compact packaged units which protect the gears and shafts and lubrication inside an enclosed housing (See Figure 17.21). In addition, the closed gearing is used as a torque equalizer, to match operating speeds and torques on various portions of the mechanical system.

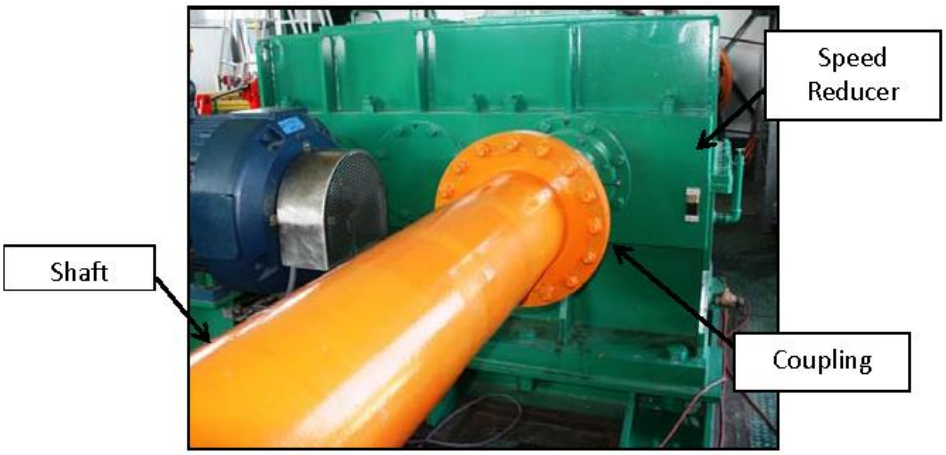


Figure 17.21 Speed Reducer (Closed Gearing), Shaft, and Coupling

- Shafts: Shafts transmit all the required mechanical power from one part of the machinery system to another for opening and closing operations. Figure 17.21 shows a shaft and coupling at the connection to the speed reducer on a vertical lift bridge. Shaft condition is directly related to the structural integrity and proper functioning of the movable bridge. Unanticipated distress on the shaft may indicate either degradation of the shaft, motor, gears, rack, or overloading of the bridge during operation. Shafts ends are usually connected by couplings, which are secured to the shaft by an interface fit.
- Couplings: Couplings transmit power between the ends of shafts in line with one another (See Figure 17.21). Some types of couplings can be used to compensate for slight imperfections in alignment between the shafts. Couplings are manufactured in

large variety, but can be categorized into three general groups: rigid, flexible, and adjustable. Rigid couplings are commonly found on older bridges and basically clamp shaft ends together. Flexible type couplings are designed with internal elements that allow for misalignment during operation due to distortion of the bridge structure to avoid bending the shafts. They also simplify shaft installation by allowing slight misalignment at the joints. Adjustable couplings are in span drives of connected tower lift bridges and other drives that require angular adjustment with time because of wire rope stretch, etc. They are also used to precisely rotate electrical control devices such as limit switches to prevent damage.

- Bearings: Bearings provide support and help to maintain alignment of rotating shafts, trunnions, and pins. An example of a bearing which is supporting a span drive shaft is shown in Figure 17.22.

Figure 17.22 Bearings Supporting Span Drive Shaft

- Brakes: Brakes on movable bridges are generally used as a “parking brake”. In other words, they are engaged when the span has stopped moving (much like the parking brake in your car). Brakes can be of either the drum type or disc type, and can be released manually, electrically, or hydraulically. An example of a drum type brake assembly is shown in Figure 17.23. They are generally spring applied for fail safe operation. Motor brakes are located close to the drive to provide dynamic braking capacity, except that brakes are to stop the movable portion of the bridge and to hold it in place. The manual, mechanical and electrically actuated brakes may be the drum or disk type. If they are used as machinery brakes, they are typically located close to where the fixed and moving parts of the bridge meet. Dynamic brakes typically serve as motor brakes, and are located close to the motor unless the motor is equipped with internal dynamic braking.

Figure 17.23 Typical Brake Assembly

- Air Buffers and Shock Absorbers: Air buffers and shock absorbers are located between the span and the pier at points where impact may occur between the two that mechanically control the deceleration of the span at the fully closes and/ or fully open positions. Typically the buffers are large pneumatic cylinders, or hydraulic energy absorbers. Major components of air buffer cylinders are the cylinder housing, the piston, the piston rod, inlet and outlet piping, and the strike plate. Gravity (or spring pressure for horizontally mounted buffers) extends the piston rod, which fills the void in the cylinder with air. As the span lowers, the rod is pushed in, causing the air inside to be compressed. A pressure relief valve allows the air to escape beyond the pressure setting. Forces are required to build-up and keep the pressure of the air at the movement of the span for a “soft” touchdown on the bearings. Shock absorbers achieve the same purpose as the air buffers. However, they are self-contained and, therefore, require little maintenance. An example schematic and photos of air buffers are shown in Figure 17.24 and Figure 17.25, respectively.

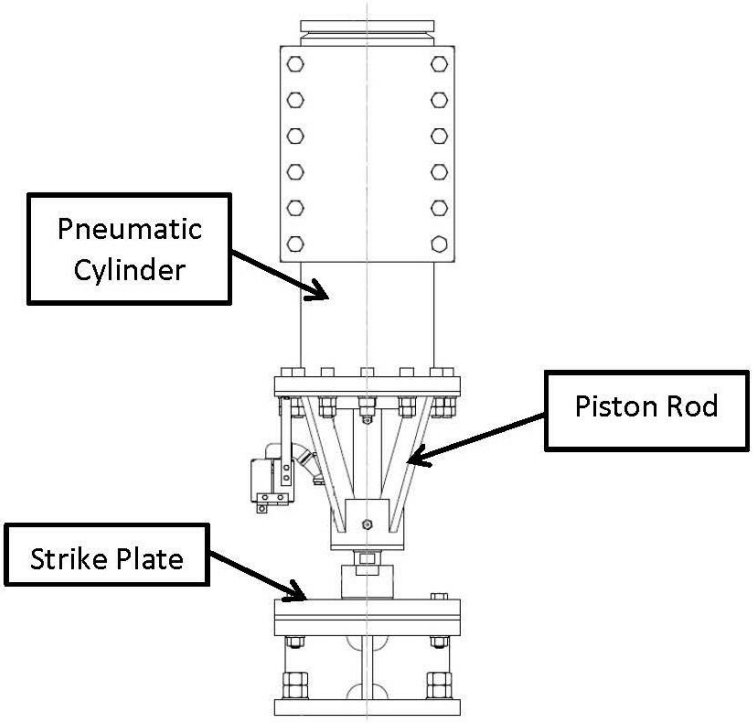


Figure 17.24 Air Buffer Schematic

Figure 17.25 Air Buffers in the Retracted (left) and Not Retracted (right) Positions (right - Courtesy of MaineDOT)

Span Locks: Span lock bars at the end of the span are engaged when the span is fully closed to prevent movement under live load. Span locks may also be provided at other locations on the span to hold the span in an open position against strong winds or to prevent movement from

an intermediate position. They can be engaged either mechanically or hydraulically and typically consist of a forged steel lock bar that engages a receiving socket. Sockets on the drive end of the lock bar serve as guides and supports to the lock bar while in operation, and when engaged. The sockets are typically welded or cast housings equipped with adjustable wear shoes. Adjustment of the spacing is made by adjusting the shims and wear shoes. The lock bar operates within the sockets by a motor and gear driven crank mechanism. Linear lock bar operators are also common. Photo examples showing span lock bars and shims are shown in Figure 17.26 and Figure 17.27. The housings are usually mounted to the side of the bascule girders, or in the webs of the floor beams. Span lock couplings need to be loose enough to allow a proper opening operation, but at the same time, the gap between the bar and the receiver has to be small enough to ensure the adequate connection with minimal bouncing, under live load, from one leaf to the other. Deterioration or incorrect operation can cause a failure of the lock to retract or engage, triggering sensors and disruption of the operation of the bridge.

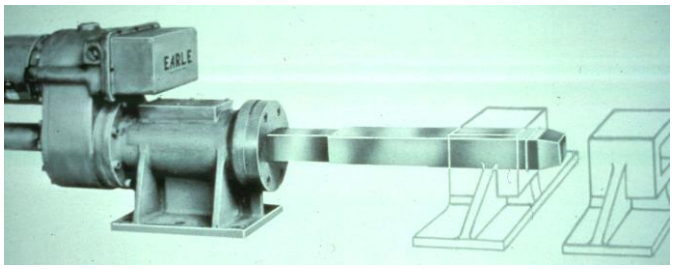


Figure 17.26 Mechanically Driven Span Lock

Figure 17.27 Span Lock Bars and Shim Removal (Courtesy of FDOT)

And example of a disengaged span lock with labelled components is shown in Figure 17.28.

Figure 17.28 Disengaged Span Lock (Courtesy of Indiana DOT)

An example of an engaged tail lock is shown in Figure 17.29.

Figure 17.29 Engaged Tail Lock

Additional examples of a tail lock in fully engaged and disengaged positions are shown in Figure 17.30 (left) and Figure 17.30 (right), respectively.

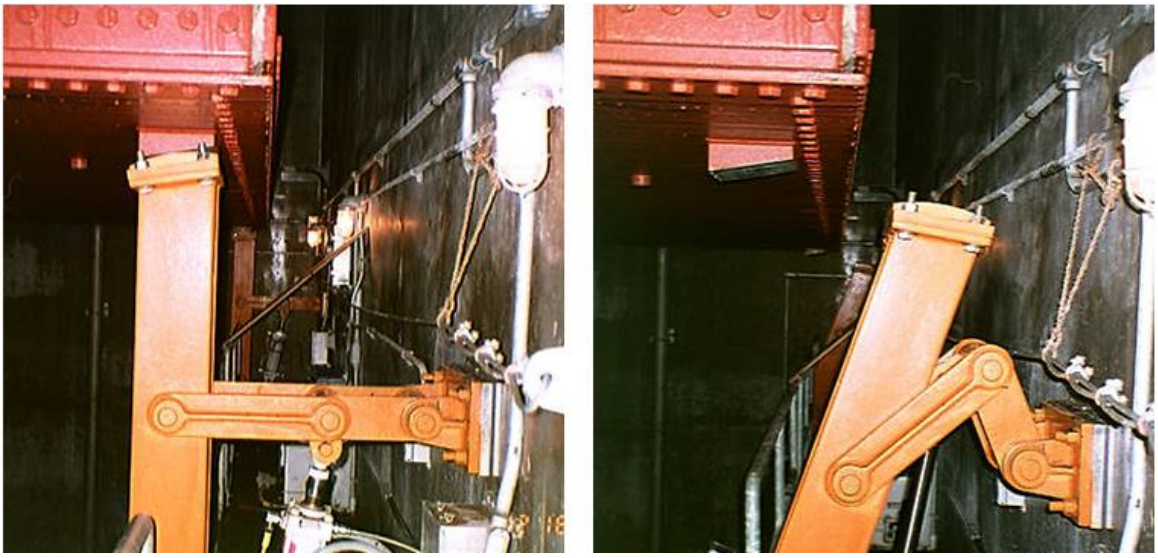


Figure 17.30 Tail Lock in Fully Engaged Position (left) and Disengaged Position (right) (Courtesy of MaineDOT)

Toe Locks: Toe locks transfer vehicular live load between the leaves of double-leaf bascule bridges when the structure is in the closed position. Unlike tail locks where the bridge can still carry vehicular traffic without the span engaged in the seated position, double bascules cannot carry traffic if the toe locks are not engaged, as the continuity of the two bascule cantilevers is required. A toe lock typically consists of a forged steel lock bar that engages a receiving socket. An example of a toe lock that is still not fully engaged is shown in Figure 17.31.

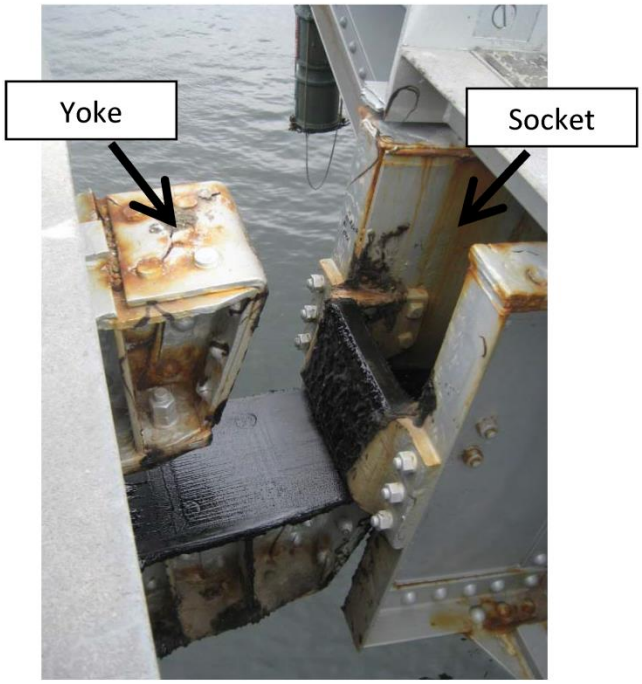


Figure 17.31 Toe Lock on Double Bascule Bridge

- Live Load Bearings and Strike Plates: Live load bearings and strike plates between the movable and fixed portions of the bridge are designed to bear most or all of the live load when the bridge is carrying traffic. Live load bearings are provided at the end of the

moving leaf truss/girder at the approach end. An example for a lift bridge is shown in Figure 17.32.

Figure 17.32 Fixed Live Load Bearing

- Wire ropes: These ropes are typically used to connect the vertical lift span to the counterweights located in the towers for operation on vertical lift bridges and are comprised of individual wire strands that are tightly wound together. The wire rope is terminated at the lifting girder and counterweight using open spelter sockets affixed to each end (see Figure 17.33). Rope clips and wedge sockets are often used for terminations at the operating drums. An example of a drum and wire rope on a vertical lift bridge is presented in Figure 17.34.

Figure 17.33 Wire Rope Socket Terminations

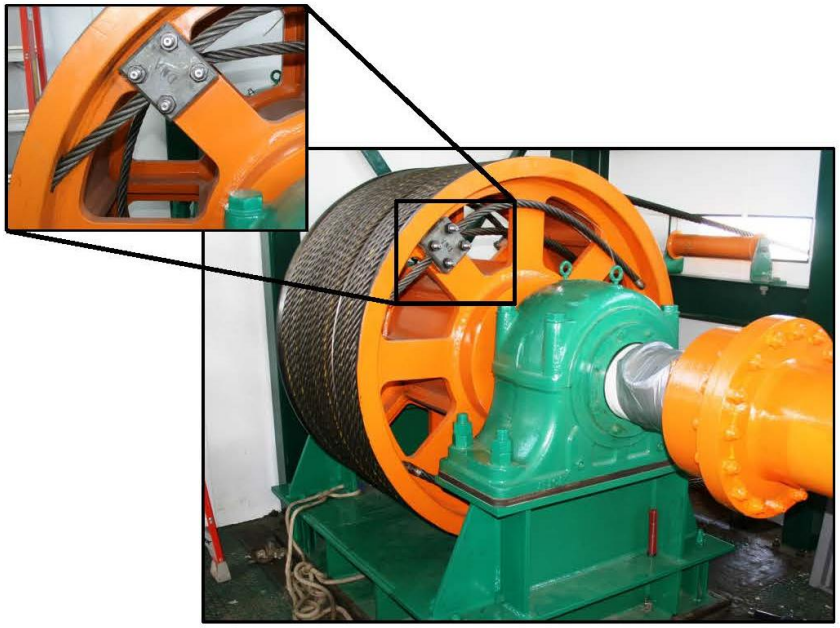


Figure 17.34 Operating Drum and Wire Rope with Rope Clip on Vertical Lift Bridge

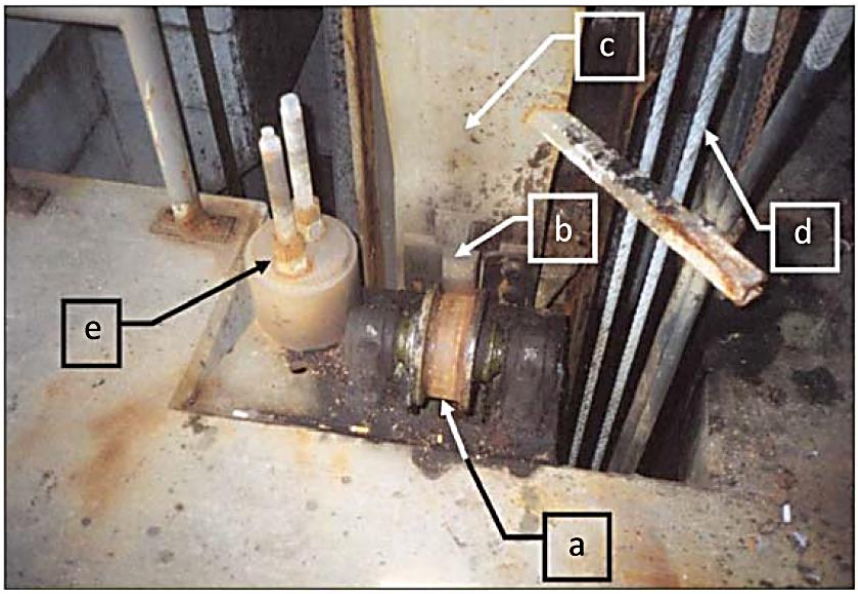
- Counterweight Sheaves: These are large diameter, angular-grooved wheels located at the top of the towers, over which the counterweight ropes are draped, supporting the entire weight of the span and counterweight during operation. These sheaves are supported by shafts and bearings. An example of a counterweight sheave is shown in Figure 17.35.

Figure 17.35 Counterweight Sheave

- Operating Drums: Operating drums are large diameter, grooved drums to which the operating ropes are attached and are paid-out and taken-in during operation of the lift

span. These drums are supported by shafts and bearings that are turned by the operating machinery. For Span Driven vertical lift bridges, these operating drums are located on the lift span. Drums are located at the tops of the towers on Tower Driven vertical lift bridges. The drums are grooved in a spiral fashion in order to store several wraps of wire rope. The grooves are cut to a specific dimension to properly space adjacent ropes and support the rope without crushing it.

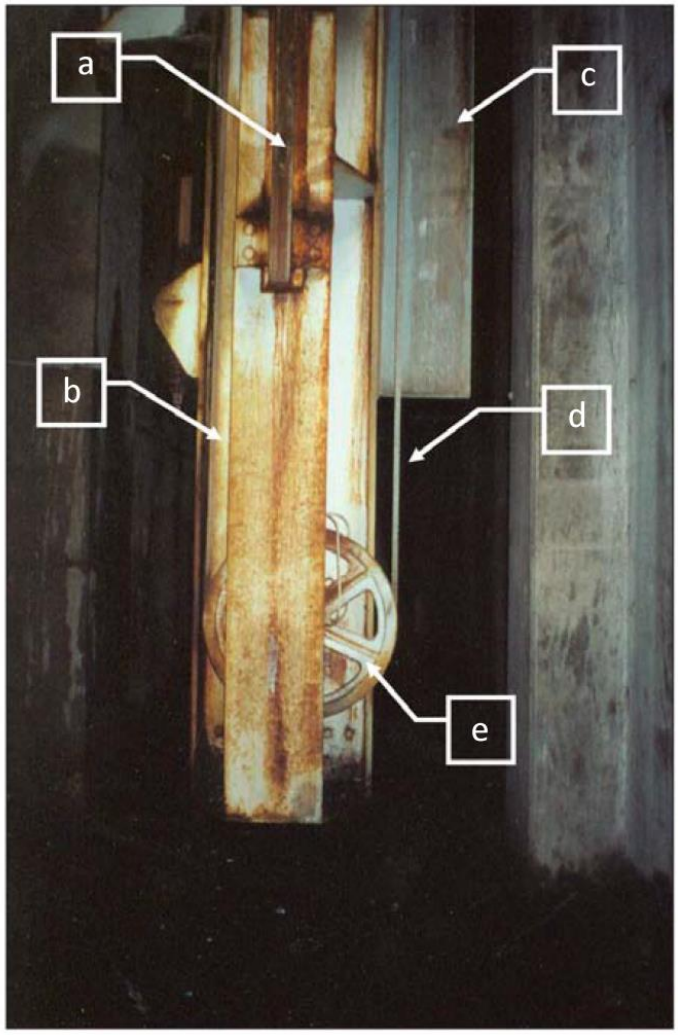
- Span Guides: Span guides restrict lateral and longitudinal movement of the span during operation. Roller guides are commonly used as span guides. Guide rails located along the tower legs are for contact with the guides against any side wind loads. An example of the guide rail detail is shown Figure 17.36.



a Guide roller
b Guide rail
c Lifting post
d Equalizer ropes
e Rope take-up

Figure 17.36 Span Guide Rail Detail (Courtesy of Wisconsin DOT)

- Counterweight Guides: Counterweight guides are used to limit lateral and longitudinal movement of the counterweights as they move within the towers. Sliding guides are frequently used as counterweight guides.
- Lifting Posts: The spans are guided vertically at each corner of the span by lifting posts, as shown in Figure 17.37. Typically, lifting posts are rigidly connected to the end floorbeams and fascia girders.



a Plate clamp rail
b Lifting post
c Guide rail
d Equalizer rope for longitudinal system
e Lower sheave

Figure 17.37 Components of a Lifting Post (Courtesy of Wisconsin DOT)

- Equalizing Systems: Skewing of the lift span during raising and lowering is restrained (in both the longitudinal and transverse directions) by wire rope stabilizing systems. The stabilizing system is separate from the counterbalancing systems described earlier. Figure 17.38 shows a moving drum stabilizing system. One rope is fastened to the pier wall and passes through sheaves then wrapped around a drum and anchored on the fixed portion of the structure. A second rope provides the same from the opposite pier wall. The drum is mounted on the movable span. An stabilizing system is necessary because center of gravities could be different between the span and lift cylinders, and there could be different degrees of friction in the listing posts.

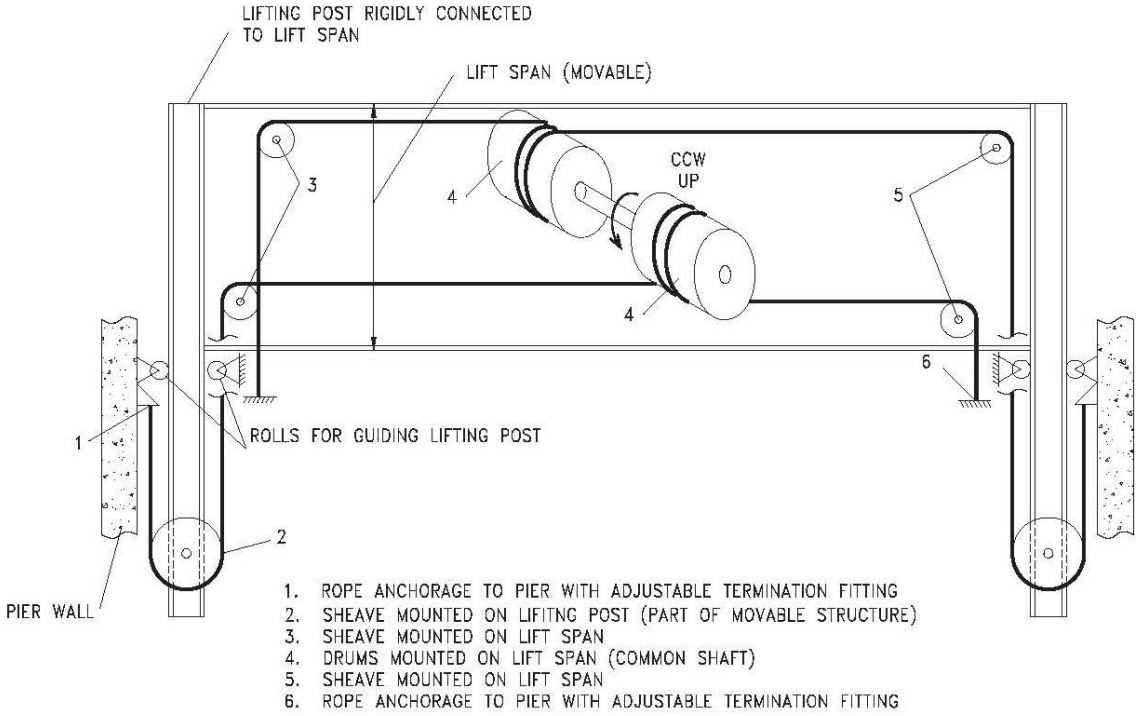

LIFTING POST RIGIDLY CONNECTED TO LIFT SPAN
LIFT SPAN (MOVABLE)
CCW UP
3
4
4
5
6
1
2
ROLLS FOR GUIDING LIFTING POST
PIER WALL
1. ROPE ANCHORAGE TO PIER WITH ADJUSTABLE TERMINATION FITTING
2. SHEAVE MOUNTED ON LIFITNG POST (PART OF MOVABLE STRUCTURE)
3. SHEAVE MOUNTED ON LIFT SPAN
4. DRUMS MOUNTED ON LIFT SPAN (COMMON SHAFT)
5. SHEAVE MOUNTED ON LIFT SPAN
6. ROPE ANCHORAGE TO PIER WITH ADJUSTABLE TERMINATION FITTING

Figure 17.38 Moving Drum Equalizing System (Courtesy of Wisconsin DOT)

- Tread Plates: Tread plates are mating steel plates with interlocking teeth and pockets on rolling bascule bridges. The upper curved tread plate, which is mounted to the bottom flange of the segmental girder on the bascule leaf, rolls along the stationary bottom tread plate, which is mounted to a frame or supporting steel. As the bascule leaf travels along the lower tread plate, the teeth mate with the pockets to ensure proper alignment and travel during the opening of the leaf. These plates see very high loads, as the entire weight of the bascule leaf is resting on it. An example of tread plates is shown in Figure 17.39.

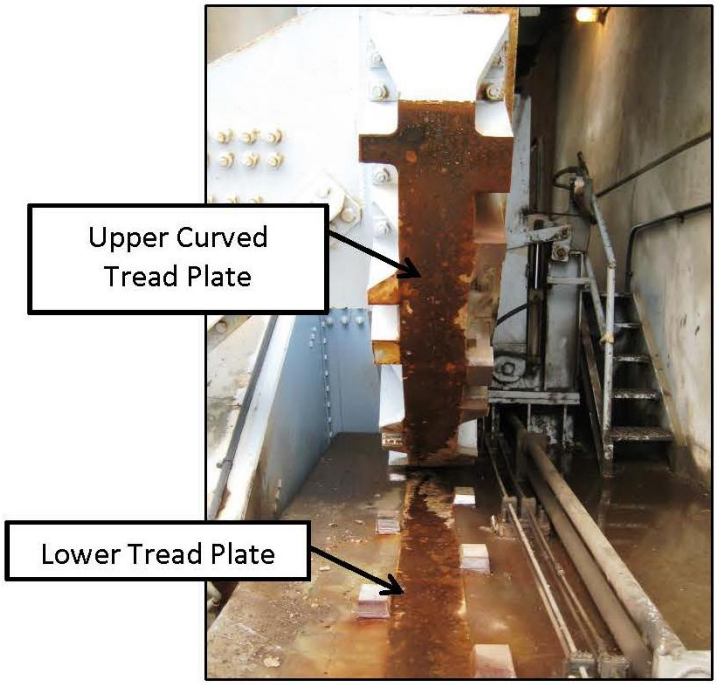


Figure 17.39 Tread Plates

17.2.2.1 Types of Span Drives

Span drives consist of an arrangement of electrical and mechanical components to transfer the high speed, low-torque energy from the motors into the low-speed, high-torque motions of the two pinions.

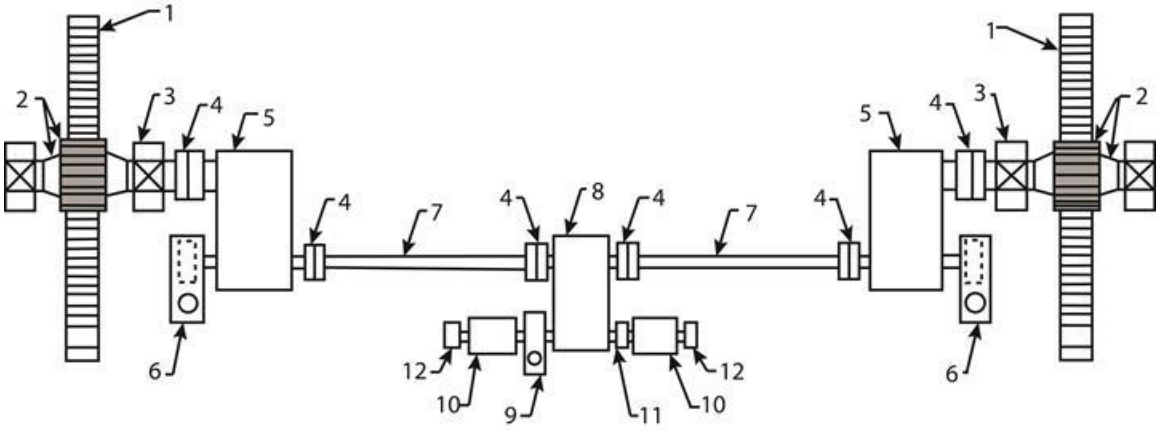
Drives may be electric motors, hydraulic equipment, or auxiliary drives. Either AC or DC power may be used to drive electric motors. AC power is often used to power wound rotor motors with torque controllers on older bridges, while new bridges may utilize squirrel cage induction motors with adjustable frequency speed control. Hydraulic equipment is usually either large actuating cylinders or hydraulic motors. Adequate pressure and volume of hydraulic fluid must be provided to the cylinder, or motor to power the opening and closing of the bridge. The fluid flow to the cylinder or motor is usually provided by an electrically operated and controlled hydraulic pump.

Auxiliary drives and emergency generators may be provided to serve in the event of power failure. Auxiliary motors and hand operators, and appurtenances, are provided to serve in the event the main drive fails. To avoid the need for larger auxiliary generators, some bridges require auxiliary motor use any time the auxiliary generators are used. The smaller, less powerful auxiliary motors will usually take longer to operate the bridge. An example of an auxiliary drive assembly is shown in Figure 17.40.

Figure 17.40 Auxiliary Drive Assembly

The pinion of the span drive machinery engages with the rack on the movable span when opening and closing the span. The pinion is the last gear in the drive train. As it drives the motion of the span, it is commonly called the drive gear. The rack can be another gear, or a bar, or a track with teeth on one of its sides. The teeth of the rack are machined to match the teeth of a pinion and move the span. The three most common types of span drives are designated as Type 1, Type 2, and Type 3 Span Drives, as discussed on the following pages.

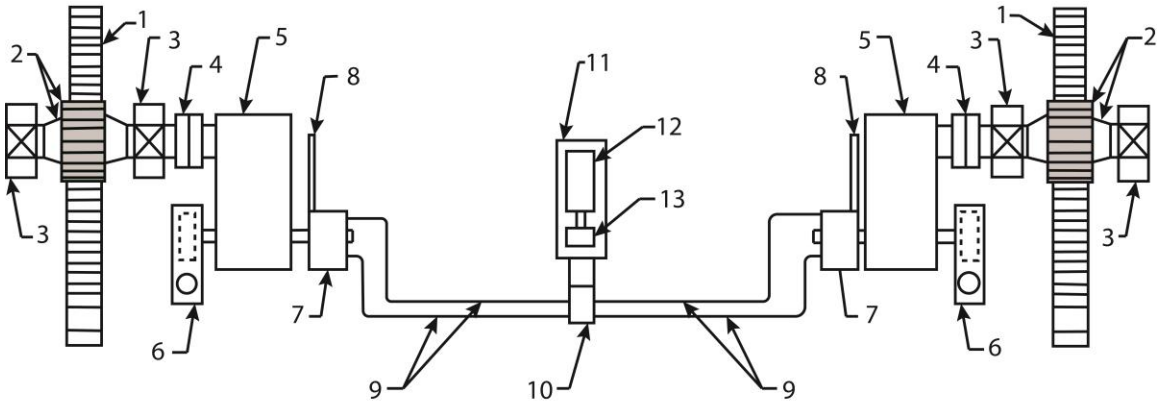
Type 1 Span drives have all the outputs connected mechanically, as shown in the example in Figure 17.41. The motor is connected to the primary speed reduction gearing, usually by a flexible coupling. This gearing is enclosed in a welded steel housing on most contemporary designs, but cast steel and various kinds of cast iron were and are used in manufacturing these speed reducers. Power is distributed from the primary reduction gearing to two (or more) sets of secondary reduction gearing located at the sides or ends of the movable span. Secondary speed reductions are often made with enclosed speed reducers as shown; however, open gearing is not uncommon. Some newer bridges may have a single large speed reducer, rather than smaller primary and secondary reducers.



1. Rack
2. Pinion Shaft and Pinion
3. Bearing
4. Gear Coupling
5. Secondary Gear Speed Reduction
6. Machinery Brake
7. Floating Shaft
8. Primary Speed Reduction
   Type 1 - ND Drive: No Differential
   Type 1 - AD Drive: Active Differential
   Type 1 - LD Drive: Lockable Differential
9. Motor Brake (With Flexible Couplings)
10. Motor
11. Flexible Coupling
12. Tach Generator

Figure 17.41 Type 1 Span Drive Schematic

A Type 2 span drive has two pinions, each powered by a separate hydraulic motor through a geared speed reduction, as shown Figure 17.42. Because the fluid input to the two motors comes from a common source, the torques applied by each pinion should be nearly equal, assuming that both sets of motors and gearing, and piping, etc., are alike. Open gear trains are also common.



1. Rack
2. Pinion Shaft and Pinion
3. Bearing
4. Gear Coupling
5. Secondary Gear Speed Reduction
6. Machinery Brake
7. Hydraulic Motor
8. Torque Arm
9. Hydraulic Lines
10. Manifold & Valves
11. Tank
12. Motor
13. Pump

Figure 17.42 Type 2 Span Drive Schematic

Type 3 Span drives transmit power to the moving span by linear action, as depicted below in Figure 17.43. Type 3 drives have been installed on simple trunnion bascules, rolling lift bascules, swing bridges, and vertical lift bridges. Hydraulic cylinders are usually used to move the span, but chain, wire rope, and screw drives have also been used. The cylinder mountings shown are for a simple trunnion bascule or a pit vertical lift bridge.

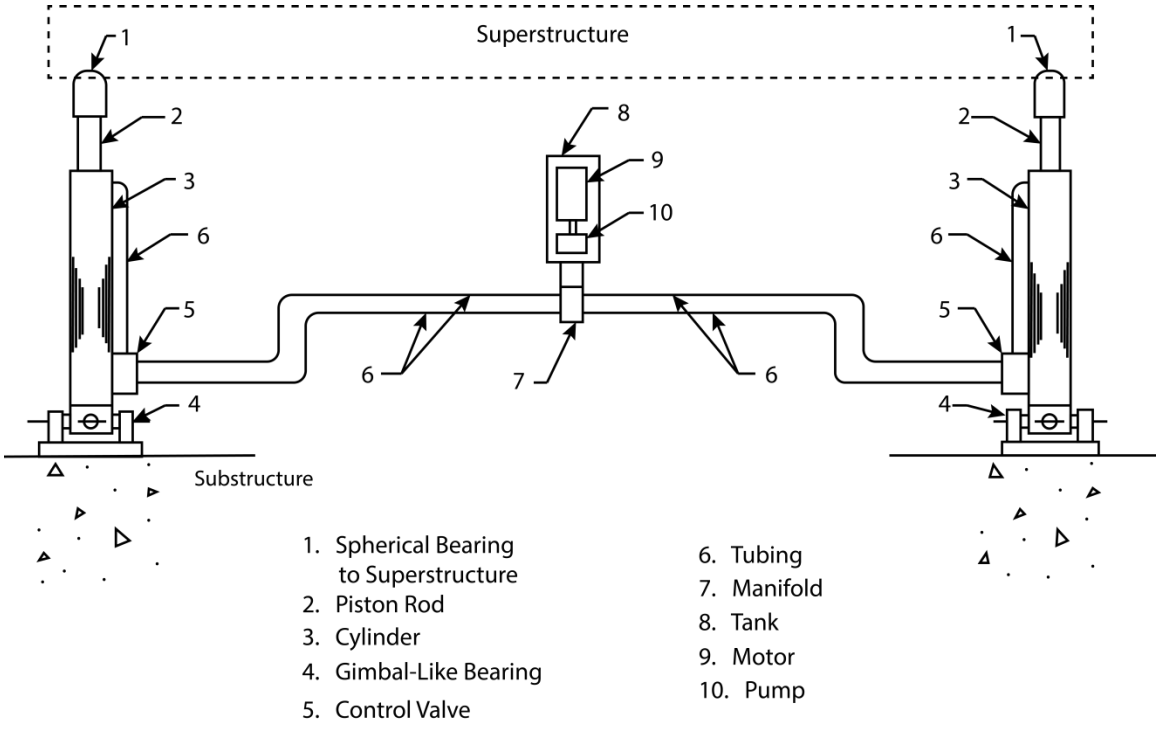


Figure 17.43 Type 3 Span Drive Schematic

The high-speed low-torque output shaft of the motor rotates a pump which pressurizes the hydraulic fluid. Modern hydraulic systems typically use mineral, vegetable, or synthetic oils for hydraulic fluid. Prior to the mid 1800's, hydraulic systems used water. Pressurized hydraulic fluid is transmitted to cylinders, which move the span directly, or to hydraulic piston motors, which move the span by rotating a rack pinion.

17.2.2.2 Open and Closed Gearing

What To Look For

- Worn gearing
- Areas lacking lubricant
- Broken, damaged or missing teeth
- Debris in gear assembly/gear assembly lubricants

Open gearing refers to machinery (gears and shafts) that is not contained in a sealed housing. The machinery are supported by shafting and bearings that are mounted onto fabricated or cast metal structural supports or framing. The gear mesh should usually be as installed, but may become misaligned due to wear or deterioration of the gears, shims, fasteners, bearings or

supports. Constant exposure to weather and the presence of abrasive, foreign materials that lodge in the gear mesh serves to accelerate the wearing of open gears.

The most common type of open gearing found on movable bridges is spur gearing. Spur gears or straight-cut gears are the simplest type of gear. They consist of a cylinder or disk with the teeth projecting radially, and although they are not straight-sided in form, the edge of each tooth is straight and aligned parallel to the axis of rotation. Spur gears transmit power and regulate the speeds of parallel shafts using gear reduction. Gear reduction allows the power from the motor shaft, which operates at a high speed but low torque, to be transmitted to the operating machinery shaft, which operates at a lower speed but higher torque. See Figure 17.44 for a spur gear example.

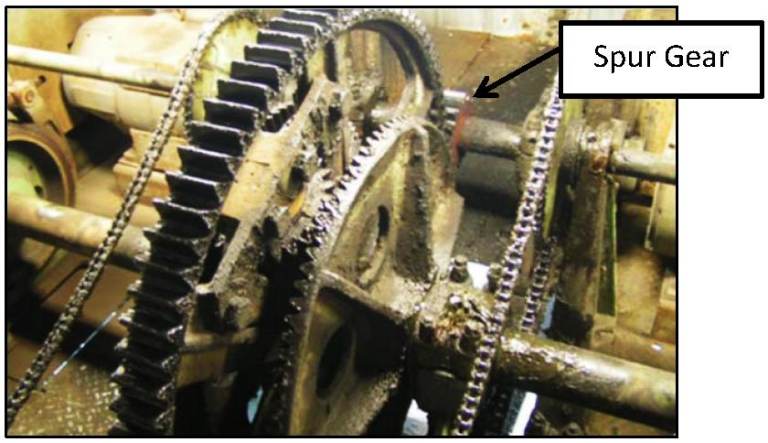


Figure 17.44 Open Gear

The rack and pinion, which are open gears, are the main gears used to operate the bascule leaf. The pinion is attached to the fixed part of the bridge, and the rack is attached to the movable leaf. The shaft delivers the force to the pinion, which engages the rack, moving it to open the leaf. For smooth operation, open gearing as well as rack and pinion gearing should be properly lubricated. An example of rack and pinion gears are shown in Figure 17.45.

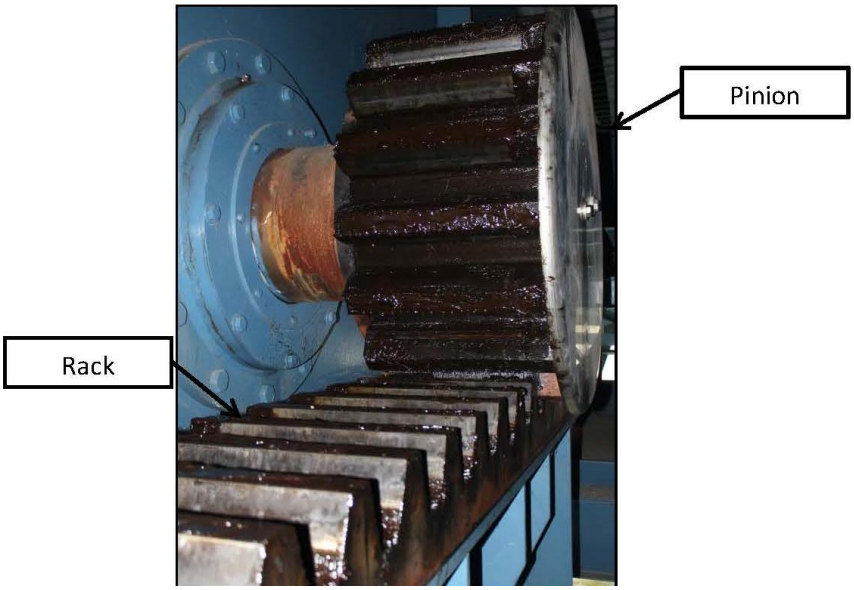


Figure 17.45 Rack and Pinion

Gear assemblies that are mounted in dust-proof, oil-tight, housings are generally referred to as closed (or enclosed) gearing, gear box or transmission. The gear reducers are often the most expensive part of the operating machinery and also one of the least thoroughly inspected components. A gear reducer is a set of gears mounted within an enclosed housing that maintains alignment and provides lubrication. The housing encloses the gear sets providing protection from dirt (and contaminants) while providing an environment where lubricant can be splashed onto the gears. An example of closed gearing is provided in Figure 17.46.

Figure 17.46 Closed Gear (Speed Reducer)

On bridges these assemblies usually function as speed reducers. Speed reducing gearing is designed to transfer the high speed, low torque of the motor to low speed, high-torque to raise or lower the span. In many newer bridges, speed reducers or transmissions are standard equipment manufactured by companies that manufacture mechanical power transmission equipment. Older reducers and transmissions made of cast iron or cast steel. Custom made reducers and transmissions on newer bridges are usually fabricated with welded steel. Reducers/Transmissions are typically equipped with antifriction roller or ball bearings. Housings of mass-produced standard reducers are still cast from various grades of iron and steel.

17.2.2.3 Bearings

Bridge bearings for non-movable bridges are covered in Chapter 12 of this manual. A bearing is typically a device that is used to enable rotational or linear movement, while reducing friction and handling stress. A simple bearing example is a car tire that rolls when loaded. The two primary loads are radial and thrust. Radial loads can be thought of as that force which causes the tire to rotate. If the car goes around a corner, a thrust force acts sideways on the tire. Using this simple example, most bearings are designed to primarily handle rolling, thrust, or a combination of each.

In general, movable bridge bearings provide support and prevent misalignment of rotating shafts, trunnions, and pins. An example of a bearing is shown in Figure 17.47. Virtually all bearings in heavy movable structures have tight inner ring and loose (or floating) outer ring fits. Tight inner ring fits eliminate shaft damage caused by fretting corrosion. Fretting is defined by the ASM Handbook on fatigue and fracture as: "*A special wear process that occurs at the contact area between two materials under load and subject to minute relative motion by vibration or some other force.*"

The loose outer ring fit allows the outer rings to slide axially to accommodate shift in the structure, thermal expansion, and mounting inaccuracies, except on fixed trunnion designs where the bearing outer ring rotates and the inner ring is stationary. In that case, the outer ring has an interference fit and the inner ring is loose. All housings should be specified as to fixed or floating: fixed is designated by the suffix "F", and floating is designated by the suffix "L".

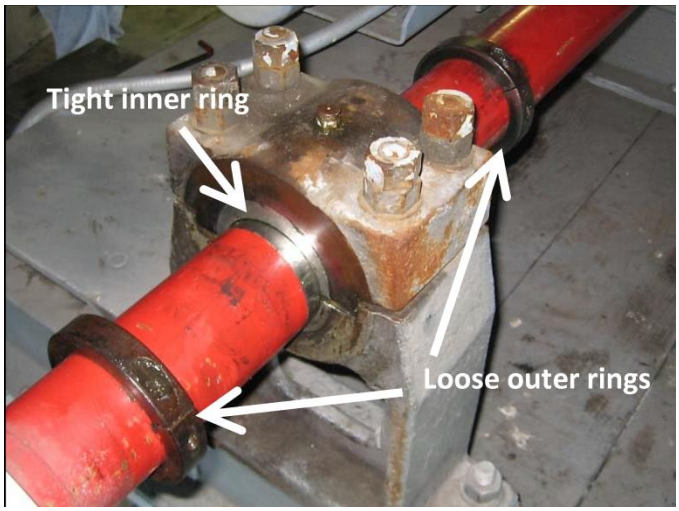


Figure 17.47 Bearing

Several specific types of bearings are discussed below, including:

- Sleeve Bearings or Plain Bearings (for radial motion)
- Roller Bearings
- Linear Bearings
- Thrust Bearings

The type of bearing used depends on many operating factors and how the bearing is going to be used in the operation of the movable bridge. For example, sleeve bearings are generally the most reliable, operate efficiently at high speed, can be repaired, are relatively inexpensive, have a long life and are overall tougher than the other bearings. Roller bearings operate more efficiently and accurately at lower speeds, require no maintenance, and their friction is not set; their friction is lower at lower speeds and higher at higher speeds.

A *sleeve bearing* (also called a *plain bearing* or *journal bearing)* is a fixed cylinder (also called the sleeve or bushing) in which a shaft rotates. The sleeve is usually made of bronze and is held to a fixed point within a steel housing. Housings are usually split in order to accommodate installation, removal of the shaft for repairs, or replacement. The top half is bolted down to the base, and the base is bolted to the structure. The sleeve bearing often has a flange that functions as a thrust surface to hold the shaft horizontally positioned.

Sleeve bearings are common in movable bridge trunnion bearings for vertical lift bridge sheaves and for bascule spans. Due to the high loads, slow operating speeds, and need for lubricant retention (no mechanical seals), grease is most always used. Their size and large bearing clearances provide for aggressive convergence zones that coat ultra-thin films of fresh lubricant into the contact area. An example of a sleeve bearing is shown in Figure 17.48.

Figure 17.48 Sleeve Bearing (Courtesy of NHDOT)

A *rolling-element bearing* or roller bearing is generally composed of a sleeve-like outer ring and several rows of balls retained by cages. The cages were originally machined from solid metal and were quickly replaced by stampings. This bearing type features smooth motion, low friction, high rigidity and long life. They are economical and are easy to maintain and replace. Plain bearings are very similar in design to rolling-element bearings, except they slide without the use of ball bearings. Heavy-duty spherical roller bearings are often used on operating machinery shafts and sheaves, as these elements see higher loads. These bearings are commonly used in trunnion bearings and sheaves. Bearings in vertical lift bridges undergo multiple revolutions in opening and closing the bridge and this increased rotation makes spherical roller bearings the primary consideration for their longer life. An example of a roller bearing is shown in Figure 17.49.

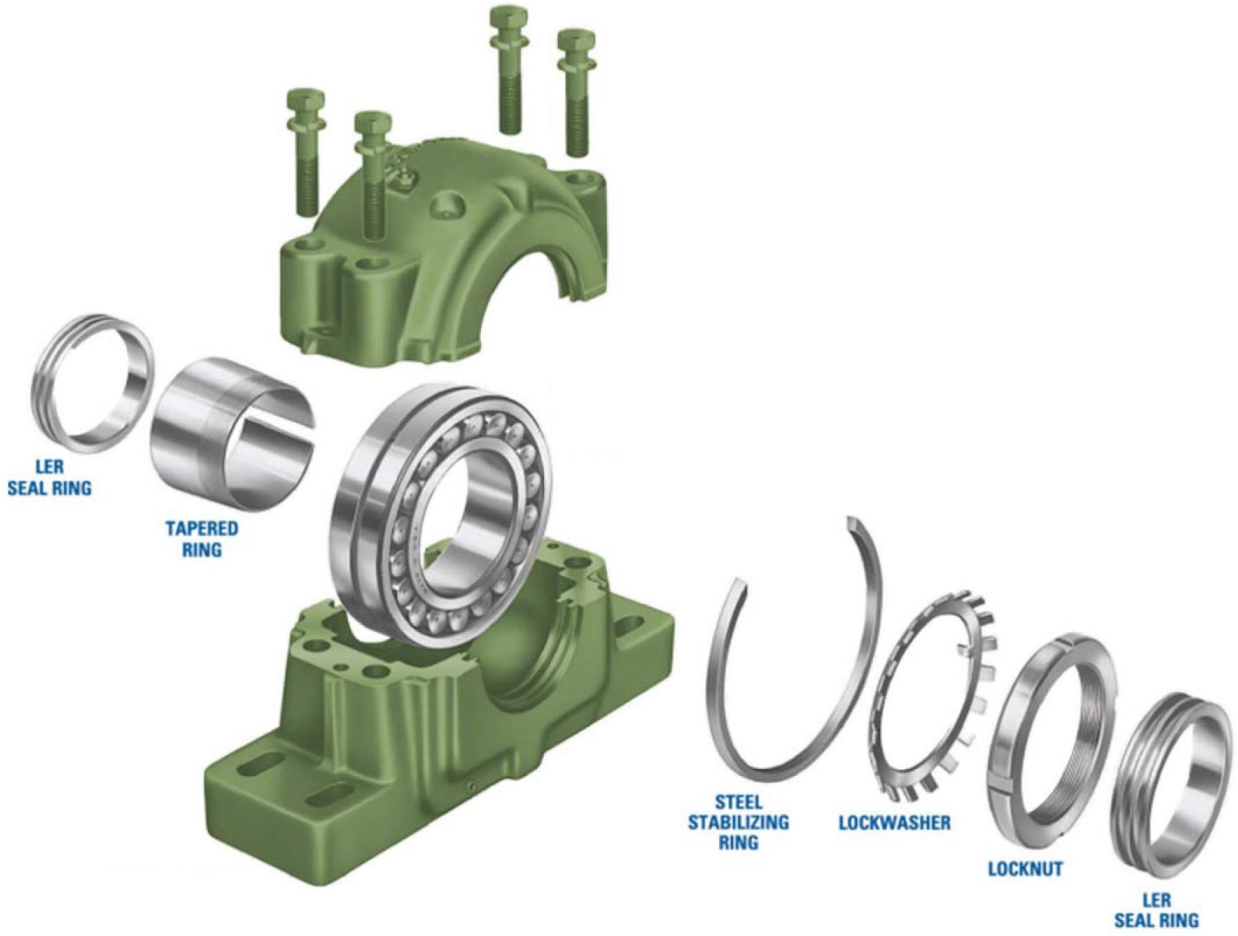


Figure 17.49 Roller Bearing (Courtesy of The Timken Company)

A *linear-motion bearing* or *linear slide* is a bearing designed to provide free motion in one direction. There are many different types of linear motion bearings. These can be rolling element bearing or plain bearing. An example of a linear bearing is shown in Figure 17.50. On a movable bridge, these serve as span guides, centering devices, lifting wedges, toe-locks, or tail-locks.

Figure 17.50 Linear Bearing

Thrust bearings are most commonly used for the larger center pivot bearing used on swing bridges. These large bearings consist of two l discs, one is bronze and the other is steel. The upper disc is a convex shaped while the lower disc is concave shaped with a slightly larger radius of curvature and may have rollers. This slight difference in curvature, amongst other things, provides for a centralized contact area at bearing center with convergence zones surrounding and immediately adjacent. Spherical roller thrust bearings have a higher contact angle resulting in increased stability and thus less sensitivity to seismic or impact loading from vessels or floating debris. An example of a roller thrust bearing is shown in Figure 17.51.

Figure 17.51 Roller Thrust Bearing (Courtesy of The Timken Company)

17.2.3 Electrical Systems

A typical electrical system for a movable bridge comprises four major groups of equipment:

- Power Distribution Equipment
- Electrical Machinery
- Control System
- Lighting Systems

The power distribution equipment includes: electric power sources, protective devices, and distribution equipment. The primary power source for movable bridges is a three-phase electric service from a local utility company. Typical three-phase electric service voltages are 120/240-volt, four-wire systems (usually found on older bridges) and 277/480-volt, four-wire systems. Some bridges may be furnished with ungrounded three-wire 480-volt electric services.

Electric power is supplied to the various motors and electrical equipment through fuses and circuit breakers which serve as protective devices. Fuses and circuit breakers provide overload and short circuit protection to the electrical equipment they serve. These protective devices are housed in panel boards, Motor Control Centers (MCC), and/or enclosed panels.

A panel board contains a group of circuit breakers to distribute power to various electrical devices. Motor control centers house circuit breakers, fuses, motor starters, motor controllers and other equipment required to control and distribute power to motors and other equipment. Motor control centers are modular in construction. In lieu of panel boards and motor control

centers, circuit breakers, fuses, motor starters, motor controllers and other motor control equipment may be installed on an enclosed panel. If enclosed panels are used, they are usually custom built for the bridge. An example of a MCC is shown in Figure 17.52.

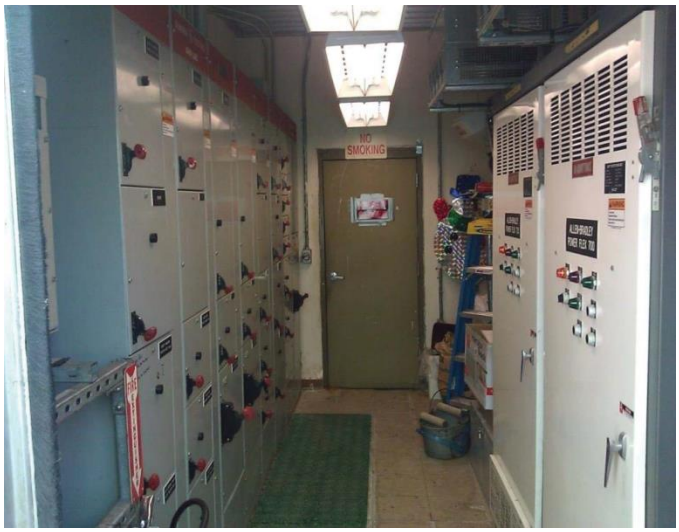


Figure 17.52 Motor Control Center (MCC) (Courtesy of NHDOT)

Electrical circuits are carried from panel boards, motor control centers, enclosed panels and transformers to the electrical devices they supply power to through a raceway system. A raceway system typically consists of rigid metal conduit and junction boxes, an example of which is presented in Figure 17.53. Electrical wires, or conductors, carry electrical current and are installed inside the conduit and boxes that make up the raceway system. Wiring carrying electricity from panel boards, motor control centers, transformers and enclosed panels travels through a raceway system of conduit and junction boxes to the devices to be powered.

Figure 17.53 Example of Electrical Conduit Work Under a Bridge (Courtesy of Kyle Asfahl)

In lieu of a second utility service, a secondary source of electric power can be provided by an engine generator set. An engine generator set utilizes a combustion engine (usually a diesel engine) as a prime mover to operate an electric generator. The generator generates an electric supply with characteristics similar to the utility electric service. An engine generator set may be

located permanently at the bridge site or it may be a mobile unit that is brought to the site when needed.

Electrical Machinery refers to electro-mechanical devices that operate the movable span and auxiliary devices such as locks, wedges, and traffic control equipment. The movable span is provided with one or more span motors that serve as the prime mover for the span. Span motors may be of the alternating current (AC) type or direct current (DC) type. Depending on the type motor control equipment employed, the operating speed of the span motors is governed by the bridge operator or by a motor controller. A motor controller provides controlled motor speed and torque to ensure smooth movement of the movable span. The electrical part of span drive is comprised of span motor and motor controller combination.

The bridge operator controls the operation of the bridge and its associated equipment with the control panel, as shown in Figure 17.54. There are pushbuttons, control switches, indicating lights, meters and indicators and often a foot pedal switch mounted on the floor at the control desk. Foot switches can be used to turn electrical equipment on and off with the foot, freeing the hands to perform other operations or providing ergonomic improvement to a workstation. Foot switch applications often require specific electrical ratings, enclosures, pedal actions, cables, and other variations. Typically, foot switches contain single-pull double-throw (SPDT) contacts that are wired "normally open." That is, the switch is “open”, or "off," when not in use.

Figure 17.54 Control Panel

Motor controllers range from simple contactors to motor drives. The equipment may be installed in a panel or motor control center. A standard motor controller consists of a motor protector, a contactor, and a motor overload device. A motor circuit protector is either a circuit breaker or a fuse that has a trip setting to protect the motor controller and motor. An example of a schematic of a motor control system is shown in Figure 17.55.

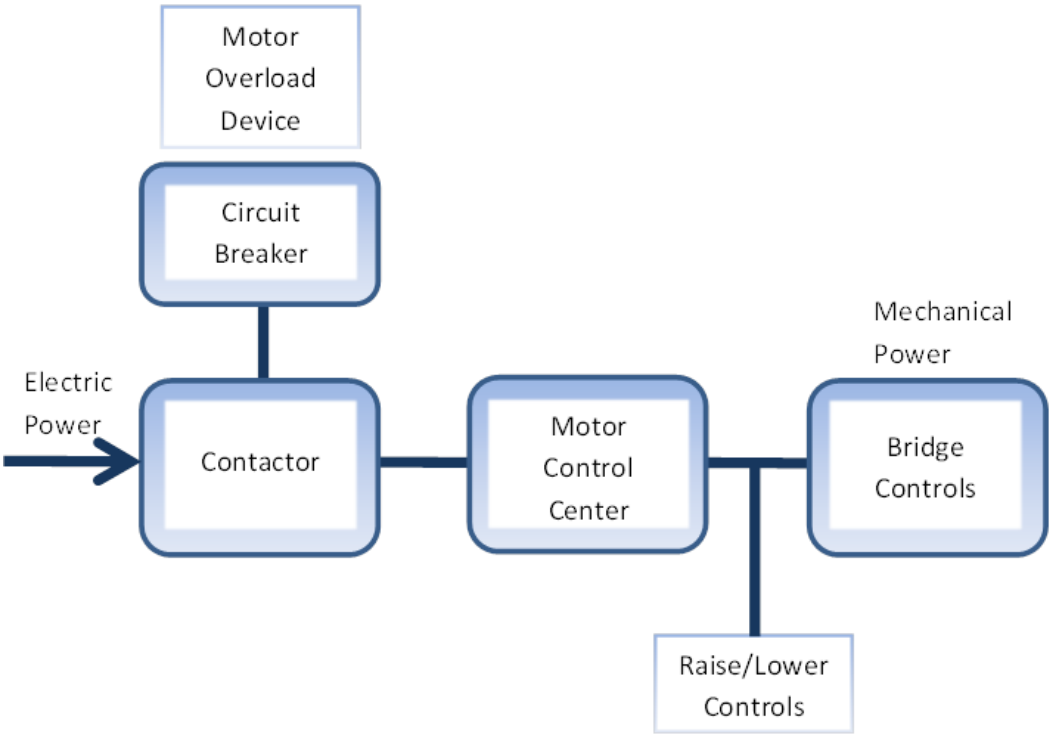


Figure 17.55 Schematic of a Motor Control System

Contactors make or break current to the motor. An example of a contactor is shown in Figure 17.56. When the motor is connected to the current it will operate. When the current is removed, it stops. Contactors may operate the motor in a single direction, or forward and reverse directions. An overload device which is operated by a switch on the MCC is a sensitive, quick acting device that will sense when the motor is drawing too much current and open the contactor to stop the motor. Motor overload devices are intended to be faster in reacting to a motor fault than a circuit breaker or fuse, and more sensitive to minor faults that would not trip a circuit breaker.

Figure 17.56 Drive Motor Contactor

Drum controllers are a common method of motor control on older bridges. Primary drum controllers are used with DC motors and secondary drum controllers are used with alternating

current (AC) wound rotor motors. The rotation of the drum controller handle changes the motors speed and direction. An example of a drum controller switch is shown in Figure 17.57.

Figure 17.57 Drum Switch to Control Motor Speed and Direction (Courtesy of Brian D. Luster)

Limit switches regulate the limit of travel for machinery and provide an electrical signal to stop or change operation. The limit switches in routine use include: lever arm, plunger type, rotary, and proximity. An example of a limit switch is shown in Figure 17.58.

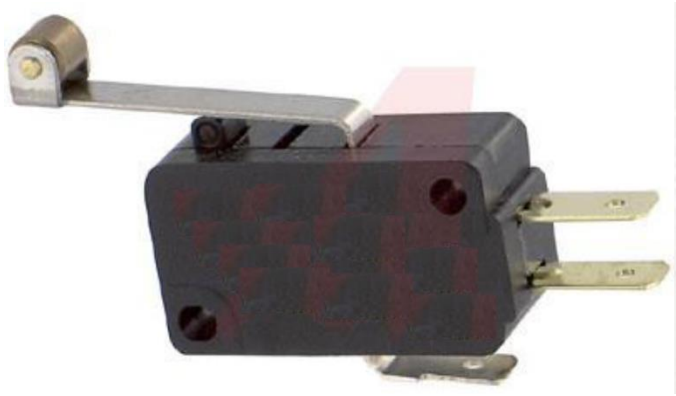


Figure 17.58 Example of a Limit Switch

Relays are low current switching devices that provide logical control of a bridge. They can be used independently or concurrently with a programmable logic controller (PLC) system. Multiple relays are required to provide control for an entire bridge where the relays are generally located in a panel or enclosure. Relays are also used as part of auxiliary systems, such as traffic gates, for control of local equipment.

Programmable Logic Controllers (PLC) processors are computer controllers that provide the logical control of the bridge. They are generally rack-mounted in cabinets in the control room. There may be multiple remote Input/Output (I/O) drops throughout the bridge, an example of which is shown in Figure 17.59. Input/Output, or voltage, drop is a measure of how the supplied energy of a voltage source is reduced as electric current moves through the passive elements (elements that do not supply voltage) of an electrical circuit. Voltage drops across internal resistances of the source, across conductors, across contacts, and across connectors are undesired; supplied energy is lost (dissipated). Voltage drops across loads and across other

active circuit elements are desired; supplied energy performs useful work. Since PLC's are computer technology, their processors can use communication networks to transmit information from a remote drop to the main processors. Small switches on the processor and I/O cards called dip switches are configured to allow proper operation. Changing these switches will alter PLC operation. Any changes to PLC dip switches should be done by qualified individuals who are familiar the PLC programing and the logic basis for the particular bridge's operation sequence.

Figure 17.59 I/O Blocks Controlled by the PLC (Courtesy of NHDOT)

Bridges are often equipped with roadway lighting to illuminate the roadway for vehicular traffic. Illumination levels from bridge to bridge may vary depending upon the speed and volume of traffic and local requirements. Roadway lights are typically pole mounted fixtures with an offset mast arm that overhangs the roadway. Roadway lighting on bridges is usually owned, operated, and maintained by a local agency.

Service lighting and receptacles are often provided throughout the structure to enable work and inspection in dark areas or during limited ambient light. Lighting fixtures have various types of bulbs. Receptacles that are exposed to the elements should be covered in compliance with current electrical codes. Service lighting and receptacles are installed to provide light and power to specific areas and equipment, not to illuminate the entire structure.

Warning gates similar to gates used at RR crossings are used to warn approaching cars and vehicular traffic during bridge operation, as shown in Figure 17.60. Warning gates alert the traffic that it must stop and indicate the vehicle exclusion area. The gates are usually equipped with flashing lights that also warn the traffic. Traffic signals, or red flashing lights, are used to initially stop the traffic at a pavement marking (stop bar). Once the traffic has stopped, the warning gates are lowered to exclude vehicles.

Figure 17.60 Traffic Barrier and Warning Gates

The lights should begin operating when the warning signals are activated and operate until the bridge is reopened to traffic. An example of warning signals is shown in Figure 17.61. They should operate from the time the traffic signals initiating bridge opening are activated until the gates are raised.

Figure 17.61 Warning Lights and Gong

Some bridges are equipped with resistance gates or barriers that can physically stop a vehicle that is either out of control or whose driver failed to realize the bridge was opening. Resistance gates are rated for a certain size of car traveling at a certain speed. They may be equipped with flashing lights. An example of a resistance gate is shown in Figure 17.62. An example of a drive motor cabinet for a resistance gate is shown in Figure 17.63.

Figure 17.62 Resistance Gate (Courtesy of MaineDOT)

Figure 17.63 Drive Motor Cabinet for Resistance Gate (Courtesy of MaineDOT)

Gongs (bells) or horns are used to alert the vehicular traffic travelling on the bridge to be alert to traffic signals. They are intended to operate at the beginning and end of a bridge operation. The traveling public expects the bridge to be in the closed position. To prevent sudden and unexpected stops, the gongs aid in making the traveling public aware that traffic conditions are changing. They are used in tandem with flashing lights and warning signs. The gongs should start operating when the warning signals are activated to stop traffic and should continue to operate until the locks, jacks, and wedges are released. They should operate again from the time the locks, jacks, and wedges are engaged until the gates are raised. The gongs should make a loud, audible sound.

Navigation lighting and signals are provided to guide and alert the channel water traffic. An example of is shown in Figure 17.64. Red lights mounted on the piers or fenders mark the channel for the boats to pass through. Alternating red and green lights mounted on the span notify the boat operator that the span is either not fully open (when the light is red) or is fully open and it is safe to proceed (when the light is green). Lights are installed in accordance with Coast Guard standards and guidelines. Proper maintenance of the navigation lights is essential for the safety of the waterway traffic. An air horn or similar audible device is used to warn the water traffic that a bridge operation is about to start. It is sounded before opening bridge.

Figure 17.64 Navigation Lights

When evaluating switchboard for maintenance needs, two checks should be undertaken. First, the lights on the control panel should be activated to verify their operation. Usually, there is a switch or pushbutton that will test all the indicator lights on the panel. Any light that operates improperly should be noted and scheduled for repair. The bridge should also be operated several times to verify that all pushbuttons, control switches, indicating lights, meters, and indicators operate properly.

17.2.3.1 AC and DC Motors

Span motors may be alternating current (AC) type or direct current (DC) type. An example of a motor is shown in Figure 17.65. Depending on the type of motor control equipment employed, the operating speed of the span motors is governed by the bridge operator or by a motor controller. A motor controller provides controlled motor speed and torque to ensure smooth movement of the movable span.

Figure 17.65 Motor

AC motor design is comprised of three windings contained in the stator, or stationary part of the motor that drives a rotating shaft, called the rotor. The AC current drives the coils, causing them to develop rotating magnetic fields. A changing magnetic field generated by AC voltage causes a rotor in the AC motor to spin around the motor shaft. An example of an AC motor is shown in Figure 17.66. The photo is a cutaway of an AC motor so that the specific elements can be identified.

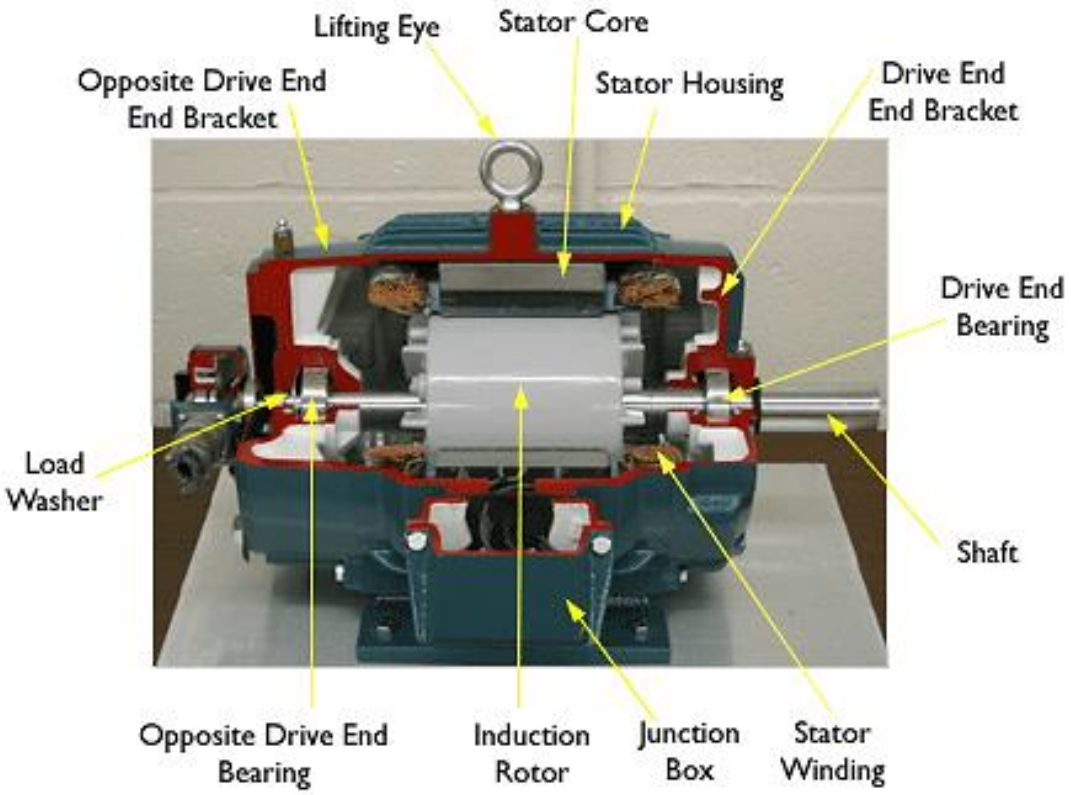


Figure 17.66 AC Motor Cutaway (Courtesy of Darby Electric Co., Inc.)

Probably the most common type of electric motor used as the prime mover in movable bridge drives has been, and still is, the AC wound rotor induction motor. Its most notable characteristics are its high starting torque and the variety of speed/torque settings (which can

be altered in the field). Most movable bridges applications that use AC motors will use either NEMA Type D or Type M motors depending on the type of speed control used for span operation.

AC motor speed control involves the use of AC inverter drives, which in turn require relatively expensive electronic parts. Typical AC motors should not operate at less than approximately one-third of the motor's base speed due to thermal reactions in the motor.

DC motor designs include a magnetic field generated by the motor stator by either electromagnetic windings or permanent magnets. In a brushed-type DC motor, a set of permanent magnets form the stator and a set of coils sit on the rotor. Direct current, switched between the coils, creates a repulsive force against the permanent magnets, turning the rotor. An example of a DC motor is shown in Figure 17.67. The photo is a cutaway of a DC motor so that the specific elements can be identified.

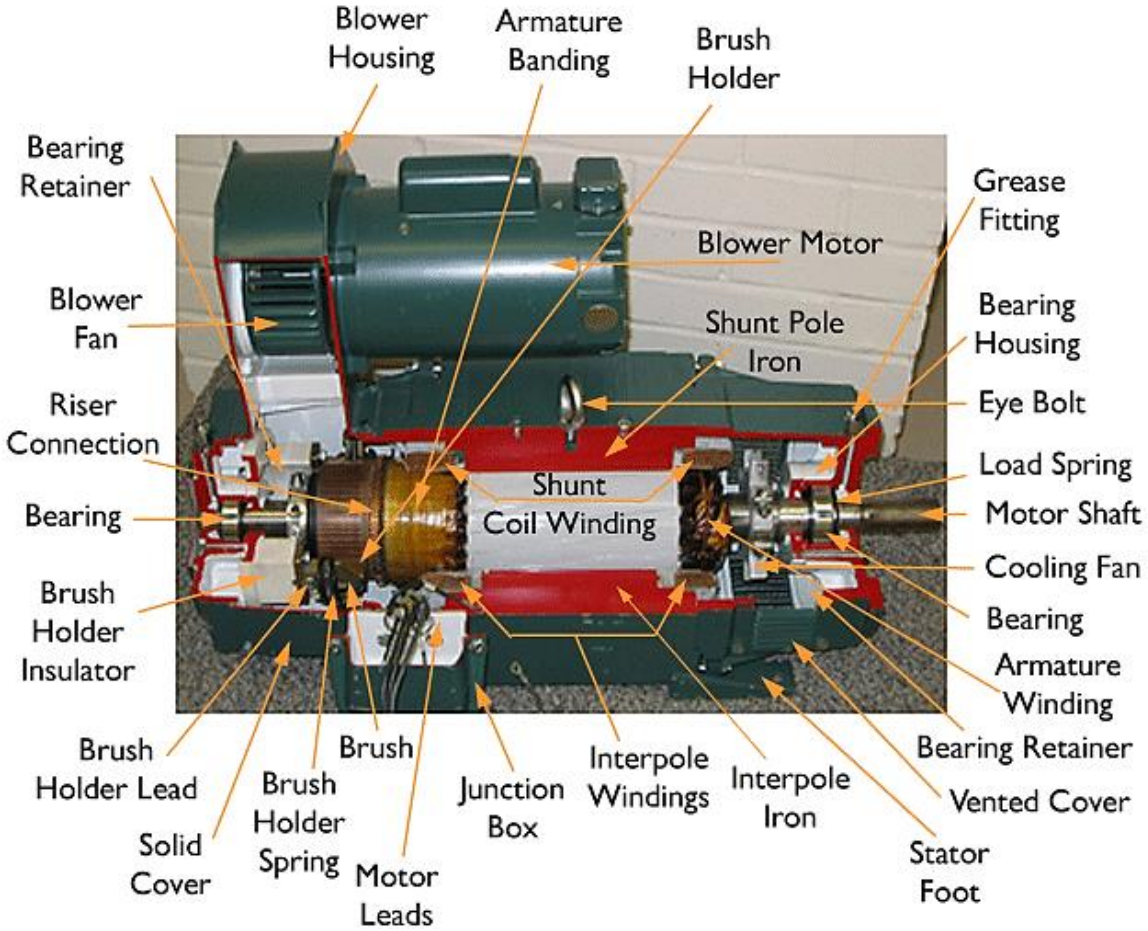


Figure 17.67 DC Motor Cutaway (Courtesy of Darby Electric Co., Inc.)

Series DC motors are used in movable bridges because of their high starting torque. They are best used for loads that require a lot of initial power to set the machinery in motion. This eliminates the need for two separate units (one to start the load and the other to keep the load moving). This type of motor is classified as a constant speed motor; constant speed motors are some of the easiest motors to control. If the load slows down, the current in the rotor goes up. Since the rotor and field coil are wired in series, the field coil current will go up too. This increases the torque of the motor and its ability to return the load to the correct speed. They

will speed up if the input voltage is increased and slow down if the voltage is decreased. DC series wound motors have a high power-to-weight ratio. This helps to reduce installation costs and provides for space savings.

Pulse width modulation is also an effective method of speed control for this type of motor. This method controls motor speed by changing the amount of time during which power is applied to the motor.

17.2.3.2 Interlock System

Sensors, triggers, and switches refer to devices designed to prevent the initiation or continuation of particular movements of the span. For instance, sensors are triggered with a traffic gate that fails to make contact with its strike plate, or if the loading becomes unbalanced, or if the wiring /conduits have failed. Examples of switches and sensors are shown in Figure 17.68 and Figure 17.69.

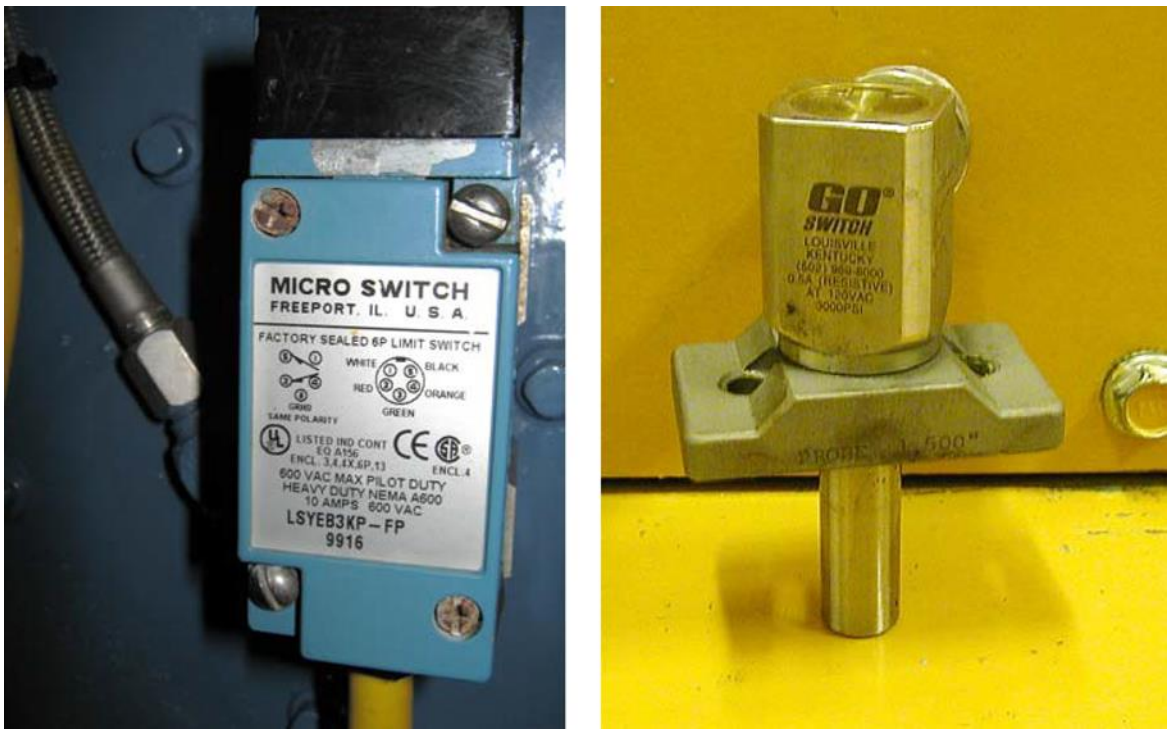


Figure 17.68 Examples of Switches (Courtesy of MaineDOT)

Figure 17.69 Limit Switch Indicates Machine Brake Position (Courtesy of NHDOT)

The bridge failing to operate is usually caused by an element of the control circuit indicating that "it is not safe to operate". In other words, a safety circuit is preventing operation. These safety circuits or "permissives" are embedded within the operating sequence of the control system, which performs checks to assure that specific actions have occurred in the proper sequence. An example of a permissive circuit for a swing bridge is assuring that the wedges have been fully retracted and the rails lifted prior to rotating the span. The devices which tell the control system that certain actions have occurred are generally limit switches. Mechanical failure of a limit switch often results from the contact arm or its support becoming damaged such that the switch does not operate mechanically. These failures are caused by corrosion, misalignment of other equipment, or inadvertent damage. Any mechanical failures should be immediately repaired. If the device failure does not cause the bridge to become inoperable (e.g., the circuit can be by-passed), the impetus to perform the repair is greatly reduced. It should be noted that by-passing safety circuits becomes an opportunity to cause severe damage and impact safety.

17.3 Preventive and Basic Maintenance of Movable Bridges

Maintenance costs for movable bridges are considerably higher than that of a fixed bridge, because (1) movable bridges have moving parts subject to additional wear, as compared to a fixed bridge and (2) by their nature, movable bridges are closer to the water and subject to more moisture than fixed bridges. Deterioration is the primary concern since they are located over waterways, and often close to the coast, which constitute conditions suitable for corrosion, causing section losses. In addition to the usual deterioration mechanisms of all bridges, movable bridges can experience operational failures. Operational failures are typically the result of misalignment, poor maintenance practices, bearing and gear wear, and trunnion scoring. One study attributed more than half of all failures were the result of improper actions taken by the maintenance staff. The improper actions were categorized as:

- Performing a temporary repair instead of a permanent repair
- Failure to install all covers and guards
- Making a system modification without documenting the change
- Incorrect diagnosis of the problem

- Bypassing of the interlock system

Deterioration and damage is also observed due to moving parts, friction, and wear of the structural and mechanical components. Fatigue can be a problem due to the reversal or the fluctuation of stresses as the spans open and close. Breakdowns cause problems for both land and maritime traffic. Maintenance staff needs to understand how the bridge systems function together so that they can identify potential equipment overload conditions. As an example, if the span is improperly balanced, the motor drives may not have the capacity to accommodate the new loads. There have been cases where back-up drives (motor controls) were not of sufficient capacity when span weights were changed.

Just as serious structural deficiencies may lead to localized failures or collapse of a bridge, mechanical and electrical defects can cause a movable span to become unstable in either, or both, the closed or open position. Serious defects of the span drive or stabilizing machinery should be reported in a manner similar to structural deficiencies. A movable span may be unstable in any position (open, closed, or in-between) due to defects in the span drive or stabilizing machinery. Much depends on the type of movable bridge and redundancy of the overall system. Because of the variations in movable bridge design, no firm rules can be set down. It is a matter of analyzing each situation.

Preventive maintenance takes the form of those tasks which must be performed to keep the movable bridges in operation. The following general tasks should be performed on a regular basis to ensure that a movable structure is in proper working order:

- Greasing of Gear Trains, Bearings, and Other Movable Parts: This task should be done often (based on the number of openings or manufacturer's recommendations for each mechanical component) to insure that the bridge lifting and locking mechanisms are properly lubricated. An inspection of the components, such as motors, gears, tracks, shafts and linkages, overspeed controls, and brakes should be done frequently. The most common causes of machinery malfunctions are the result of insufficient lubrication and a loss of bolts due to vibration. Each movable bridge should have a specific Operations and Maintenance Manual detailing the preventative maintenance items, including a detailed lubrication schedule and specifications, which should be performed on a regular basis.

Recommendation

An inspection of the components, such as motors, gears, tracks, shafts and linkages, overspeed controls, and brakes should be done frequently.

- General Housekeeping: Make sure all lubricant leaks or spills are cleaned up and that waste materials are properly disposed of. Pools of lubricants or piles of waste materials pose a trip hazard to maintenance personnel. Examine every portion of the bridge where water can collect. All pockets that are exposed to rain and snow should have a removable cover. Debris, birds, animals, and insect nests in the throughout bridge should be removed.

- Personnel Safety: Machinery and brake covers should be replaced and fastened promptly after completion of routine maintenance. Each bridge should have a site specific lockout/tag out procedure for electrical and machinery components to ensure safety during maintenance.
- Cleaning Stringers and Floor beams on Open Grid Decks: Cleaning of these superstructure components should be done in each spring using pressure waters (3,000 psi) and shovels to break up the dirt and clean the stringers and floor beams on the open deck bascule bridges. This cleaning will help reduce the amount of extra weight on the lift span to maintain proper balance, as well as reduce the amount of deterioration of the structural framing members. Certain bridges see a greater amount of debris accumulation than others; therefore these bridges may need to be cleaned more frequently.
- Cleaning of the Counterweight Pits and Sump Pumps: Pits should be pumped dry and regularly cleaned of debris. Debris and water accumulate quickly and contribute to the deterioration of the concrete pit walls and the deterioration of the steel superstructure. Where sump pumps exist, the intakes should be cleaned of debris to ensure they will not clog. All drainage elements, including the counterweight pocket weepholes should be free from debris.
- Maintenance Painting: Maintenance painting, overcoating, and total paint removal should be performed on a regular basis to prevent deterioration of mechanical and structural elements. Overcoating and maintenance painting can be done by bridge maintenance crews; however, maintenance workers should always consult an engineer before painting mechanical components to ensure that paint will not have any detrimental effects on the operation of the machinery. An example of a painted over limit switch on a brake assembly is shown in Figure 17.70. Total removal and re-coating is typically done by contractors. Refer to Chapter 18 for more information on coatings and painting.

When to Call the Engineer

Consult an engineer before painting mechanical components to ensure that paint will not have any detrimental effects on the operation of the machinery.

Figure 17.70 Painted Over Limit Switch on Brake Assembly

- Reliability Testing: Reliability testing of bridges should be done in advance of periods when there will be more frequent bridge openings to allow for adjustments to the bridge limits which insure proper operation for opening and for center and heal locks. This will help to avoid problems when there is higher demand for the bridge to be operated. At a minimum, reliability testing should be done at each biennial inspection. Gates should be checked and repaired if necessary. Live load bearings should be cleaned, shimmed, and checked for proper adjustment at the anchor columns.
- Maintaining Operational Lighting: All lighting on the bridge should be operational to ensure safe operation of the bridge and to ensure the safety of the traveling public and maintenance personnel. Replace burnt out bulbs and remove moisture from lens covers, as required.

17.3.1 Open Gears

What To Look For

- Scoring or gouges in teeth of gears
- Cracks at root of gear teeth
- Uneven grease patterns across the tooth width at the pitch line
- Excessive machinery noise during operation
- Coupling failures
- Excessive lubricant leakage
- Machinery vibration
- Deformation of teeth

The open gears are the main gears, which are part of the leaf main girder and receive the torque from the rack and pinion assembly. Excessive strain, out-of-plane rotation and misalignment are common problems for open gears. Routine maintenance is required on the gear teeth. Unless they are kept lubricated at all times, wear and corrosion due to grinding of the rack and the pinion will occur. Another concern is the loading sequence. When the drive shafts begin to advance in a delayed sequence, there can be an adverse effect on the open gears, causing impact loading. Sequence control is the method by which starters are connected

so that one cannot be started until the other is energized. This type of control is required whenever the auxiliary equipment associated with a machine, such as high-pressure lubricating and hydraulic pumps, must be operating before the machine itself can be operated safely.

The teeth of all gears should be smooth at the contact area. Scoring or deep gouges are signs of deterioration of the tooth surface. The root of each tooth should be free of cracks (this is the area of highest bending stress). An example of visible wear on gear teeth is shown in Figure 17.71.

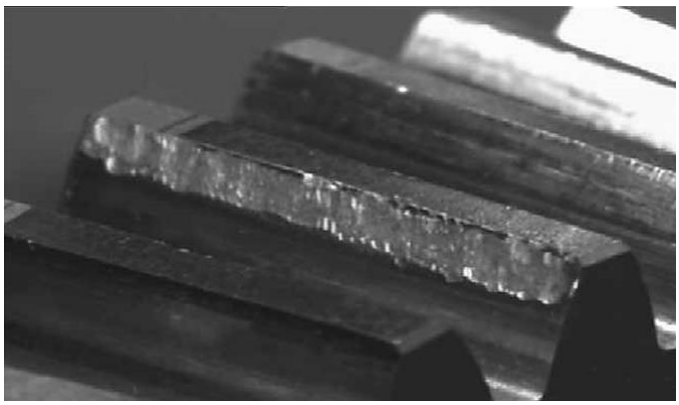


Figure 17.71 Worn Gear Teeth

Visible patterns in the grease after operation should show even contact across the full tooth width at the pitch line. (The pitch line is the ideal line in the rack and pinion assembly where the gears mesh so that the gears move in opposite direction and at the same speed). If the pattern is heavier on one edge of the tooth, or the pattern is not along the pitch line, it indicates that the shafts are not parallel to one another. If the grease patterns indicate misalignment, the degree of misalignment should be determined. Misalignment is determined by measuring the backlash between the mating gears.

In gears, alignment is defined by gear mesh, backlash and angular misalignment, or tooth contact. Bevel gears are conically shaped, as shown in Figure 17.72, and are most often matched with a rack set at an angle (commonly at 90 degrees) to the bevel gear. Bevel gears on movable bridges are commonly out of alignment. Common misalignment problems are indicated by a heavy contact at the heel portion of the tooth or at the toe portion of the tooth. Most likely, the shafting is not aligned so that the gears are at a true 90-degree angle with respect to each other.

Figure 17.72 Bevel Gear

Misalignment is easily identified in extreme situations but can be quite difficult to spot when it is less severe. Indicators of misalignment are excessive machinery noise as the machinery operates, the binding of the shafts, elevated heating or failures at the couplings, excessive lubricant leakage from the bearings, vibration of machinery components (particularly shafts), loose mounting bolts, extreme or rapid wear of gears (plastic deformation of teeth). Excessive noise can stem from a lack of lubricant or machinery that has experienced a respectable degree of wear and needs a thicker lubricant.

17.3.2 Closed Gearing (Speed Reducers)

What To Look For

- Adequate lubricant levels
- Leaking lubricants
- Abnormal leaking
- Abnormal vibration

The gear box/transmission/speed reducer contains the assembly that transmits the torque generated by the motor to the shafts. The gear box is equipped to provide the necessary amount of oil to the various gear meshes and bearings. The gear boxes should be checked for the proper amount of lubricant and signs of leaking. An example of a closed gear box (reducer) lubrication gauge is shown below in Figure 17.73, whereas an example of a leaking speed reducer (closed gear) is shown in Figure 17.74.

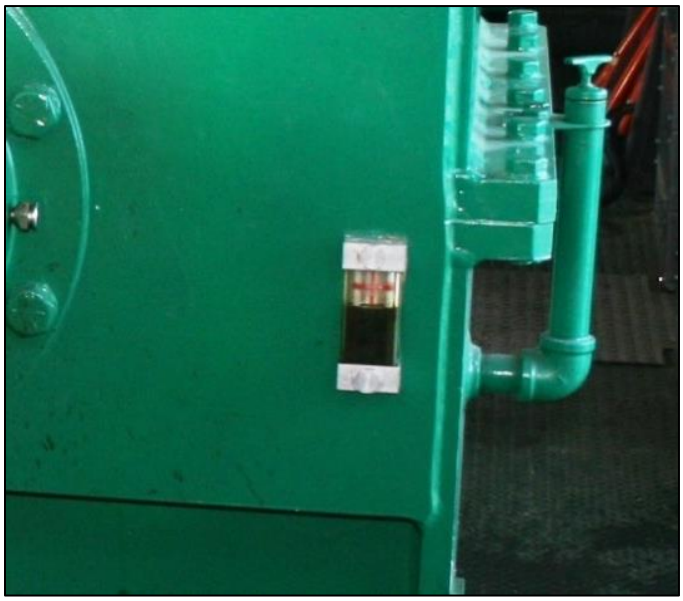


Figure 17.73 Closed Gear Box (Reducer) Lubrication Gauge

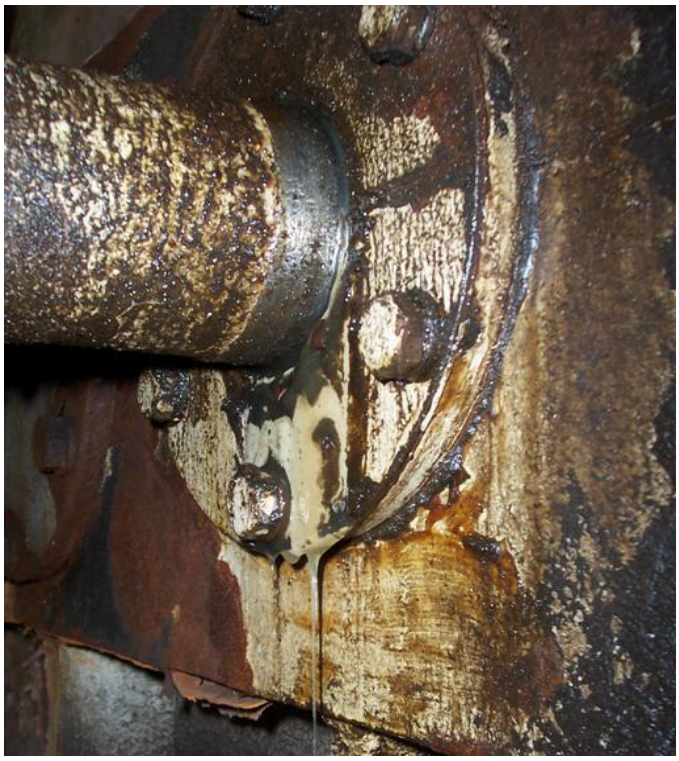


Figure 17.74 Leaking Speed Reducer (Closed Gear)

When the speed reducers experience deterioration and/or lack of lubrication, some change in the vibration and sound characteristics during the bridge operation may be heard. Abnormal vibration is an indicator of wear in the gears. The viscosity of the oil is an important parameter for proper functioning of the gearbox. The manufacturer's recommendations should be followed for changing the oil.

17.3.3 Shafts and Couplings

Shafts undergo significant twisting and strain and are subject to misalignment with other parts of the machinery system. Cracks, if suspected, may be detected using Non-Destructive Evaluations (NDE) such as magnetic particle or dye penetrant.

The coupling hubs, housings, and bolts should be free from leaks and in good condition. See Figure 17.75 for an example of a leaking gear type coupling.

Figure 17.75 Leaking Gear Coupling

17.3.4 Bearings

What To Look For

- Warm or hot bearings. If bearings get warm or hot during operation, it may be an indication of inadequate lubrication or bearing wear.

A common mechanical component to fail on movable bridges is the bearings. Gears fail because their bearings failed. Mechanical failures are generally detected by abnormally loud operating levels, component failure (gears or shafts), or irregular/intermittent operation. The most common cause of gear failure or abnormal wear is misalignment. Gear misalignment results from bearing clearances becoming excessive.

Bearing clearance is the gap between the shaft and the bearing liner. A rule of thumb for this clearance is 0.001 inches per inch of shaft diameter. For example, a 6 inch shaft should have 0.006 inches of clearance between the shaft and the bronze bearing liner. This clearance can be easily measured by the use of feeler gauges. As bearings wear, this gap increases, thus changing the alignment of mating gears and/or couplings. For a bearing to achieve optimum life it requires a lubricant that sufficiently separates the rolling surfaces. The effectiveness of the

lubricant is primarily determined by the degree of separation between the rolling contact surfaces. To form an adequate film, the lubricant must meet the specified minimum viscosity at the bearing operating temperature. This minimum required viscosity is primarily determined by the bearing size and rotating speed. Lower speeds require higher minimum viscosities.

The bearing housings, pedestals, and supports should be free from cracks and leaks. The bolts in housings and those used for anchors should be tight and free of corrosion. Grinding noises and vibration can be caused by the lack of lubricant.

In sleeve bearings, bushings should be inspected for damage and excessive wear. Sleeve bearings require lubrication at the sliding surfaces of the shaft and the bushing. Normally one or more grease fittings are located at the top of the housing. A path is drilled through the housing and bushing and meets with grooves machined into the surface of the bushing. The groove is usually in a spiral pattern, which helps to lubricate the entire surface of the journal. During lubrication, confirm that old grease is squeezed from the exit ports. Clearance between the shaft and the bushing should be measured using feeler gages. The bearing housing should remain cool to the touch during operation. Any heat generation may indicate improper lubrication or damage to the bearing.

Typically heavy-duty roller bearings are used to transmit power. An example is shown in Figure 17.76.

Figure 17.76 Heavy-Duty Roller Bearing on Bascule Bridge

Lighter duty ball bearings are commonly used for instrumentation that drives electrical control feedback devices. In general, the clearances of the bearing are set during installation, and the unit is sealed for operation. Little wear occurs at these bearings, so wear measurements are not required. Indications of potential problems or failure of roller-type bearings are overheating, unusual noises and shaft or bearing vibration. Contributing factors to failure include too much or inadequate lubrication, dirt, rust or foreign materials in the bearing, a faulty ball or roller, seal failure, and loss of clearance or preloading. Antifriction bearings should operate smoothly and quietly during operation.

Sleeve-type bearings and roller-type bearings should not leak oil; an example of a leaking bearing is shown in Figure 17.77. Check the seals, mounting bolts, and housing for leaks, loose connections, cracks or damage. The oil level should be checked using the external level gage or the portal in the housing. Fill the reservoir as required with the specified lubricant. During operation, listen for any abnormal noises. The bearing housings should not be hot to the touch.

For spherical roller bearings grease is recommended in virtually all bridge applications. For the main trunnion bearings for bascule bridges and the main sheave bearings for vertical lift bridges, greases with a calcium sulfonate thickener are recommended. This thickener provides excellent corrosion and water resistance as well as extreme pressure protection under boundary lubrication. Greases that use molybdenum sulfide should be avoided since this additive can become corrosive in the presence of moisture. For spherical roller thrust bearings used for the center pivot bearing in swing bridges, oil lubrication is generally recommended since the bearing can be filled entirely with oil without relying on seals to retain the oil.

Figure 17.77 Leaking Sleeve Bearing

In general, maintenance of the trunnion bearings (see Figure 17.78) is the same as for bearings. The alignment of the trunnion is critical to prevent premature wear of the trunnion bearings and to reduce out-of-plane web distortions that introduce fatigue damage. Misalignment in the trunnion axis can cause additional load on the trunnion-hub assembly and distress on the main girder plate, causing distortions, and eventually web buckling. Trunnion misalignment is also a major cause of wobble that can result in mismatch of the leaf tips, disturbing regular operation. In-depth assessment involves disassembly of the bearing caps and dye-penetration, magnetic particle, and/or ultrasonic testing of the shaft fillet by a trained professional.

Figure 17.78 Example of a Trunnion Bearing (Courtesy of MaineDOT)

17.3.5 Brakes

Braking devices should be checked for the proper setting of the braking torque and to ensure for the complete release of the brakes when actuated. An example of a braking device is shown in Figure 17.79. Brake drums and shoes wear out, corrode, become misaligned and stick. Hydraulic disc brake systems should be free from leaks. Check the brakes, limit switches, and stops (cylinders and others) for excessive wear and slip movement. Observe the surface of the brake drum for indications of contact with the brake shoes. Check the pressure developed by each disc brake power unit to be sure the brakes are releasing. Also check the manual release on all of the brakes.

Figure 17.79 Brake Assembly Exhibiting Moderate Brake Dust Accumulation

Brakes should be equipped with covers, to prevent debris or grease from affecting brake operation. Grease, oil, water, and dirt in the brake drum will reduce the braking capacity and facilitate corrosion. Heaters or heated enclosures can also be installed to eliminate corrosion caused by condensation. During a bridge operation, time the length of operation for the brake to fully release and the brake to set while monitoring the brake shoe and drum. If the shoe and drum are not aligned, they will come into contact during operation. This contact could produce smoke and damage the brake.

If the brakes are not operating in the correct timing sequence, wear can increase dramatically. The brakes generally specified are designed for duty well beyond what is experienced on a movable bridge. In other words, when they wear out it is because of either mis-adjustment or a problem with the sequencing of when they are applied. A properly adjusted brake should have even clearance between the brake shoes and the drum when the brake is released. This clearance depends upon the brake drum size (check with the manufacturer) but can be easily checked with a feeler gauge. If the brake drum surface is corroded, then the coefficient of friction between the pads and the drum will not be the intended value. The brake torque setting should be checked every few years such that they are properly adjusted. See Figure 17.80 for a brakewheel that exhibits excessive wear grooves.

Figure 17.80 Close-up of Brakewheel Showing Significant Wear and Deep Grooves

17.3.6 Air Buffers

Air buffers should be checked for corrosion of the exterior housing and the connectors. Additionally, the buffer pistons should be observed on a regular basis to make sure they descend during operation of the bridge. The pressure in the buffers should be checked periodically to ensure that there are no leaks in the piping and that the buffers are providing adequate absorption during seating of the span.

17.3.7 Locks

The toe locks and tail locks (if used) on double-leafed bascule spans, and the end locks on single-leaf bascule bridges, swing bridges, and vertical lift bridges should not have excessive deflection. Span locks are some of the members that often fail due to deterioration or incorrect operation. Check the locks for fit and for movement of the span or leaf (or leaves). Check lubrication and for loose bolts. Verify that the lock housing and its braces have no noticeable movement or misalignment. The paint adjacent to the locks will have signs of paint loss or wear if there is movement. Check lock bars, movable posts, linkages, sockets, bushings, and supports for damage, cracks, wear, and corrosion. An example of a corroded span lock is shown in Figure 17.81.

Figure 17.81 Span Lock Rusting due to Aged and Contaminated Lubrication

Check all rear locks in the withdrawn position for clearance from the path of the moving leaf as it opens and for full engagement when the leaf is closed. Examine the gap, if any, between the locking bar and the receiving slot for sloppiness. Adjustments may need to be made if the gap does not allow for smooth operation. If the locking unit is hydraulically driven, check for leakage of oil and operation for correct length of movement of the lock.

On bascule bridges, see if the live load bearings fit snugly. Also observe the fit of tail locks at the rear arm and of supports at the outer end of single-leaf bridges.

Examine actuators for operational characteristics, including leakage if hydraulic. Note both the quantity and quality of the lubricant. Check for alignment, and analyze the type of wear that is occurring.

17.3.8 Steel Counterweights and Auxiliary Counterweights

The counterweights should be inspected for corrosion of the steel components, especially the connections to the other mechanical components (pin plates at the connections to the counterweight ropes on vertical lift bridges and the connections to the girder/truss leaves on bascule bridges). Steel members that pass through or are embedded in the concrete of the counterweight are subject to corrosion. An example of section loss of a counterweight support member is shown in Figure 17.82.

Figure 17.82 Section Loss Repair of Steel Frame Connection with the Counterweight

Additionally, concrete counterweights should be checked for excessive spalling that could affect the span balance of the structure. Most counterweights have hollow pockets or areas where extra concrete balance blocks or metal plates are located to provide additional counterweight. These areas should be inspected periodically to ensure that there is not accumulation of water and that there is not excessive deterioration of the concrete balance blocks or metal plates. Counterweights should move freely with no obstructions.

17.3.9 Live Load Bearings and Strike Plates

Live load bearings and strikes plates should be checked regularly for corrosion and should be free of excessive debris accumulation. Inspect live load supports periodically for proper alignment and proper bearing during seating, see Figure 17.83 for nonbearing support. Improper bearing of the live load supports could mean that the span is not properly balanced and may lead to overstress of the other bearing components. Expansion live load supports should move freely when bridge is not in the seated position. Figure 17.84 shows a frozen live load bearing on a vertical lift bridge.

Figure 17.83 Fixed Live Load Support Bearing Not Fully Engaged in Seated Position

Figure 17.84 Frozen Expansion Live Load Support

17.3.10 Wire Ropes

The wire ropes that support the counterweights and lift span on vertical lift bridges should be greased regularly as part of a routine maintenance schedule. Additionally, the wires should be inspected periodically for corrosion or broken strands. The wires should also be checked for wear or flattening of the strands caused by contact with the sheaves or operating drums during operation. Overloading of the movable span can lead to accelerated wear and flattening of the wire rope strands, as shown in Figure 17.85.

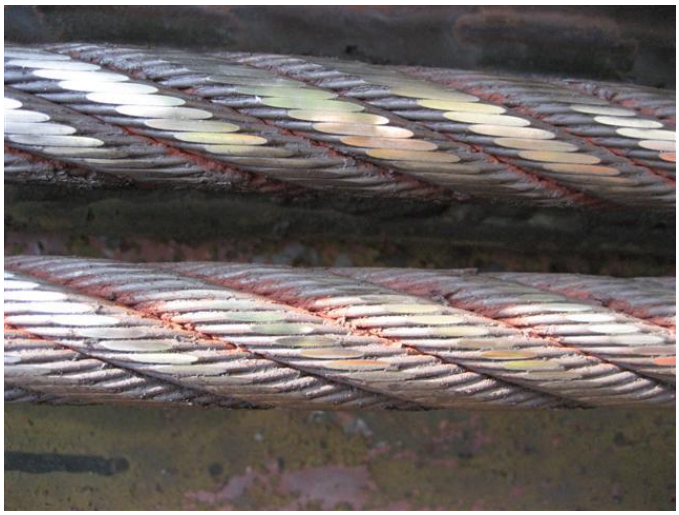


Figure 17.85 Flattened Wire Rope Strands

17.3.11 Counterweight Sheaves and Operating Drums

Counterweight sheaves and operating drums should have adequate lubrication and should be cleaned and re-lubricated on a regular basis. Any corrosion or wear should be addressed and debris removed. Check the sheaves and drums for indications of rubbing between ropes and between ropes and grooves.

17.3.12 Span Guides

Lubricate span guide rollers regularly to ensure proper rotation along the span guides. Check rollers periodically for flat spots or wear of the roller. Span guides should be should be checked periodically for wear, which can be a sign of an improper alignment of the span or lack of proper lubrication. Figure 17.86 shows a span guide roller that has been painted over so that the free rotation has been impeded. Note that all span guide rollers may not always be in contact with the guides. For instance, lateral span guide rollers on vertical lift bridges will only come into contact with the guides under a heavy lateral (wind) loading.

Figure 17.86 Painted Over Span Guide Roller Impeding Free Rotation

17.3.13 Tread Plates and Racks

Tread plates and racks should be inspected periodically for deterioration and cracks due to overstress and for overall alignment. Special attention should be paid to the gear teeth, as these elements see the highest stress. An example of worn teeth on a rack is shown in Figure 17.87.

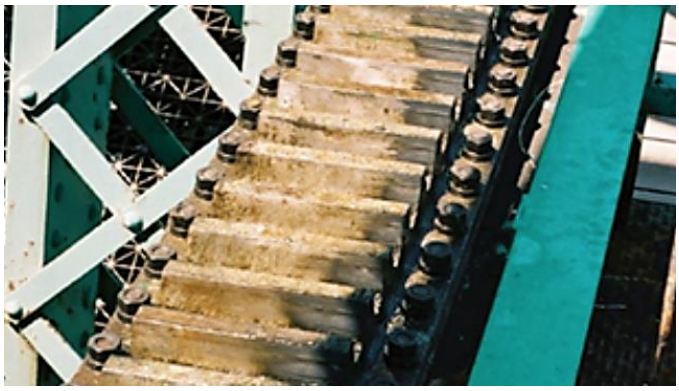


Figure 17.87 Worn Teeth on a Rack

Figure 17.88 and Figure 17.89 show a tread plate pocket and corresponding tread plate tooth on a Scherzer bascule bridge that has become worn and rounded along the edge due to a misalignment of rack relative to the tread plate. Ensure that elements are properly greased to prevent corrosion and grinding of the components.

Figure 17.88 Excessive Wear of Curved Tread Plate Pocket

Figure 17.89 Excessive Wear of Tread Plate Tooth

17.3.14 Traffic Barriers

The traffic barriers should be maintained regularly to ensure proper operation, as these elements are essential to public safety. Ensure that elements requiring lubrication are properly greased and that broken or damaged elements are replaced. Cabinets housing electrical wiring should be kept clean and free of water and debris. Replace all burnt out or broken lighting components.

17.3.15 Electrical System

The electrical system for a movable bridge is quite complex. When a short circuit occurs, the electrical circuit for the bridge is effectively changed to an unknown configuration. Electrical equipment tends to fail for several reasons: the equipment is overloaded due to a mechanical or structural failure, the insulation of conductors has broken down resulting in a short circuit, loose connections have caused an overheating condition, or the equipment is past its useful life.

One of the most common failure modes on movable bridges is insulation failure resulting in short circuits. This situation usually occurs due to wear, breaks, or corroded conduit which exposes the conductors to heat and moisture. Dirt and debris can trap moisture which leads to the breakdown of insulation. A preventive maintenance action is to vacuum out all electrical cabinets on an annual basis. Insulation quality can be tested using a megger insulation test. This test needs be carefully used as current is passed through the conductors. This test also requires that the bridge electrical system is de-energized (effectively closed to marine traffic). These tests are appropriate for large conductors where a failure would cause an extended outage.

Figure 17.90 Twisted Droop Cable with Exposed Wiring

Vibrations that occur on a bridge will cause the terminals to loosen over time. Loose terminals will cause heating in the wires and reduce the useful life of the equipment. Compression terminals traditionally have the least resistance to vibration and the wires will sometimes become loose and even fall off the terminal strips. Ring-tongue terminals provide the best resistance to vibration, as the compression screw travels through the ring on the cable. If the screw becomes loose, the ring will still maintain contact. Examples of compression terminals and ring-tongue terminals are shown in Figure 17.91.

Figure 17.91 Examples of Compression Terminal (left) and Ring-Tongue Terminals (right)

An example of a ring tongue terminal assembly is shown in Figure 17.92.

Figure 17.92 Ring-Tongue Terminal Assembly

When a broken conduit is found, it needs to be repaired immediately (repair does not include electrical tape nor wire nuts). Loose connections result in greater resistance through the connection which results in heating of the connection. The heating and cooling cycle results in the connection becoming looser until it fails or overheats causing a small fire. Ongoing minor maintenance will maintain safety circuits in a state of good repair, significantly reducing failures or interruptions. When corroded or broken electrical conduit is not immediately repaired, moisture will accumulate and the wire (conductor) insulation will deteriorate until a short circuit occurs. Short circuits on movable bridges are very dangerous because the bridge controls can effectively become rewired into an unknown configuration (causing equipment to operate when it is not intended to operate). It is not uncommon for a short circuit in one location to

cause failures throughout the control circuitry. Figure 17.93 shows an aerial cable junction box where the aerial cables have slipped down inside box several feet placing stress on the conductors and terminal strips. Using recommended electrical conduit and connections will prevent this type of problem.

Figure 17.93 Aerial Cable Junction Box Where the Aerial Cables have Slipped

The electrical motors generate the torque required for the opening and closing of the bridge. Some of the indicators for improper functioning of the electrical motors are high amperage, high temperature, high vibration level and high revolution speed. The operation of each motor should be observed during opening. Motor shafts should be checked to determine if there is normal end play. Motors should be examined to verify that they are smooth running and free from vibration. Motors and bearings should be checked for overheating. Any unusual noises generated during operation should be noted. If the motor is fan cooled, check for proper operation of the fan and that the motor is being properly cooled.

Brushes in electrical motors are subject to friction and wear. They eventually wear out and need replacement. Wound rotor alternating current (AC) motors, synchronous AC motors, and direct current (DC) motors use brushes to carry current to rotating parts of the motor. All brushes should be inspected for wear. Each brush holder should be set so the face of the holder is approximately 1/8 inch from the collector-ring. When removing brushes, be sure to reinsert them into their original holder and in its original orientation. Marking on one side of the brush before removing it will ensure it is placed back in the proper location and alignment. If the remaining portion of any brush is less than 1/4 inch or less, all brushes on the motor should be replaced. The entire surface of the brush that rides on the collector-ring should display a polished finish – this indicates full surface contact. If evidence is observed that indicates that a brush is not making full contact over its entire surface, the springs that push the brushes against the collector-rings should be inspected. Improper spring pressure may lead to collector-ring wear or excessive sparking. In this case, the springs should be replaced.

Warning: High and medium voltage cables require special protection and must be de- energized before inspecting. Only personnel trained on such equipment should perform the inspection. Before performing any inspection of the electric service contact the utility and arrange for power to be disconnected.

Specialty cables are installed in areas that cannot be serviced by wire in conduit. These include flexible cables and submarine cables. Flexible cables are cables routed between fixed portions to movable portions of the structure. Cables from the pivot pier to the swing span, cables from the rest pier to the bascule leaves, and cables from the tower to the lift span all require flexibility. The benefits of flexibility are offset by reduced jacket protection. The softer, more flexible, cable jackets that bend and move with the bridge tend to wear more quickly from rubbing and abrasion.

Submarine cables are routed into the channel through the water from one side of the bridge to the other. The cables are typically trenched into the bed of the waterway. Submarine cable is typically manufactured with a steel armor wire wrapping and polyethylene covering to protect it from the harsh conditions. The portions of the cables that are near the waterline and exposed to fluctuations in water level are usually the first areas to fail.

17.3.16 Hydraulic System

"Open" and "Closed" loop hydraulic circuits are both used on movable bridges. Open loop hydraulic circuits are the most common in movable bridge applications. Most hydraulic movable bridges are actuated by hydraulic cylinders that require large volumes of fluid and use differential flow rates in and out of the hydraulic cylinders. Open loop systems take up relatively large space due to the large volume of oil required.

In open loop circuit applications, the pump draws fluid from a reservoir and pushes this fluid into the hydraulic system. After passing through the control valve circuitry and the actuator, the fluid returns to the storage reservoir. The reservoir typically is sized to hold at least three times the volume that can be displaced by the pump in one minute. Open loop systems move fluid in one direction only. Open loop pumps normally have a large diameter low pressure inlet port and a smaller high pressure outlet port.

In a closed loop circuit design a single hydraulic pump is used to drive one or more hydraulic motors. The fluid that passes through the actuator is returned directly to the low pressure side of the pump. The pump receives the same quantity of oil at the inlet as it is pumping through the outlet. Pressure, flow, and directional control are all achieved by the controlling elements of the pump.

The advantages of the closed loop system are that high horsepower systems are compact, they require a minimum amount of excess fluid storage and they are highly efficient. The pump controls direction, acceleration, deceleration, speed, and torque of the hydraulic motor actuator, and hence separate pressure and flow control components are not needed. These systems also provide excellent dynamic braking control, which is desirable in most movable bridge applications.

There are various types of hydraulic system controls employed to regulate the amount of oil and the pressure. The three common types are called "constant horsepower control", "electronic proportional control", and "hydraulic cylinder control". Each is described below.

Constant horsepower control (also known as horsepower limiting) uses an electric motor to drive the pump at a constant speed. System controls sense the pressure in the system and regulate pump flow accordingly. If the pressure is low, the flow is increased. This type of system uses a system pressure relief valve. Once constant horsepower is achieved, the slightest

increase in system pressure will open the relief valve and bypass the minimum pump flow back to the tank.

Electronic proportional control utilizes a proportional solenoid to vary the pump displacement. As DC current is applied to the proportional solenoid, the solenoid pushes the pilot spool with a specific force. When the current, and therefore the force, is high enough the pump begins stroking and producing flow. Further increases in signal current will increase the pump's output flow proportionally.

Hydraulic cylinder control utilizes a hydraulic adjusting cylinder and pilot system to control pump flow. The pilot circuit is designed to meet the flow requirements. This type of control can be used in both open and closed loop hydraulic circuits. An example is shown in Figure 17.94.

Figure 17.94 Low Speed, High Torque Hydraulic Motor

The relatively low usage of movable bridge hydraulic systems combined with the high reliability of modern hydraulic system valves means that valve problems should be infrequent. However, dirty system fluid or poor adjustment could cause problems with hydraulic valves. Adequate and proper system filtration is the most important aspect of maintaining any hydraulic system. The operating environment of movable bridge hydraulic systems makes proper system filtering critical.

Modern movable bridge hydraulic systems typically utilize either standard mineral oil or environmentally friendly synthetic vegetable oil as their working fluid. The useful life of these fluids is limited. Factors that influence the life of hydraulic fluids include system usage, operating temperature, system cleanliness, and water intrusion. Synthetic vegetable oils are likely to oxidize after several years and require replacement. The fluid should be tested annually to determine viscosity, contamination levels, wear elements and water content. Most commonly this can be done with a visual inspection. This will also help in determining when the hydraulic fluid requires replacement.

The abrasiveness of tiny particles wearing the close tolerance surfaces of internal components will cause degradation failure of hydraulic system components. Degradation failure spreads throughout the system, and is usually not detected until the damage is irreversible. Indication that degradation failure has occurred include Sluggish system response, loss of speed adjustment accuracy, inability of the system to produce full load, and/or overheating, may indicate degradation system failure.

Catastrophic failure is the immediate failure of a system component and usually results from large particles causing moving parts to jam or stick. In pumps, dirt can block lubrication passages and cause pump failure. Large debris can collect in orifices which supply oil to the pilot circuit of pilot operated relief valve, pressure compensated flow controls, etc. Because contaminants are a leading cause of failure, proper system filtration is critical to the long term performance of the hydraulic system.

Plumbing systems for hydraulic movable bridge machinery often consist of complex arrangements of high pressure piping, stainless steel tubing, and hoses. Piping runs usually contain many bends, elbows, fittings, and mountings due to the complex nature of the bridge structure. Vibration from operation of the equipment, vibrations from vehicular traffic, as well as movement of the equipment itself has the tendency to loosen piping fittings and piping supports. This in turn can cause system leaks as well as damage to the plumbing from not being adequately supported.

Any damaged, nicked, or worn hoses should be replaced immediately. Any loose fittings should be tightened, including the mounting hardware and hangers. O-ring or other seals should be replaced as required or on schedule. Check for leaks around the manifold interface. Replace seals and O-rings as necessary. The pressure settings on all relief valves should be checked and adjusted as necessary.

17.3.17 Predictive Maintenance

Predictive maintenance that can be applied to movable bridges includes:

1. A measurement and record of the clearance for all bearings in a movable bridge machinery system
2. Repeat these measurements every year and compare the values recorded.
3. When the measured clearance exceeds a nominal value, remove shims from the bearing to obtain the proper clearance.

In existing installations, misalignment can be detected by monitoring the wear of bearings and gears. Since misalignment causes shaft bending and hence excessive bearing loads, abnormal wear in sleeve bearings is easily measured. The backlash in gear sets can also be measured and recorded. If these measurements are recorded over time and compared, a pattern will emerge. Movable bridges are machines and should be considered as such. A simple technique to monitor alignment is to annually measure and record the backlash. This approach can be used for other components, such as span locks, bearings, and sheave grooves, as well.

A dial indicator is used to check ring gear backlash in a ring and pinion gear set. It is used to measure distances and to check flat surfaces like the engine flywheel for run out. It is also used to check shaft end play. Dial gauges are used to measure back and forth movement in different areas of an automobile: brakes, engine, transmission, etc.

Backlash between the ring and pinion gear is set after the pinion gear is installed and adjusted. Side bearing preload is adjusted while performing the backlash check. After these bearing preloads have been adjusted, it is good practice to check the gear tooth pattern. Use the appropriate dye or marking compound to complete this task.

Both the side and pinion bearings are adjusted by changing the shim thickness on each side of the case. Some differentials have adjusting nuts at each end that can be turned to adjust preload and backlash. Be careful whenever working with a differential case, they are heavy, and can slide out of the differential housing easily. Always use manufacturer's specifications when making these adjustments.

More sophisticated inspection techniques, such as vibration monitoring for motors, can also be incorporated into a predictive maintenance regime.

What To Look For

Alignment can be checked through measurement and recording of backlash.

It is important to remember that the path to equipment failure is often predictable. This path can be measured such that an accurate estimate to failure can be made.

17.3.18 Structural Health Monitoring

Structural Health Monitoring (SHM) applications have gained a considerable attention as an objective methodology to evaluate the condition and performance of the structures. The Florida DOT has investigated the use of SHM for movable bridges. Electronic sensors to measure strain, displacement, acceleration, and rotation can be used to detect deficiencies in the performance of structural, mechanical and electrical components of movable bridges early and allow for proactive assessment and maintenance actions. Some investigators have explored the possibility of incorporating imaging and optical devices and combining them with sensing technology. At this time, there are only a few and limited attempts of real life testing and implementations of these ideas.

17.3.19 Generator Maintenance

Test the generator by operating the bridge under generator power as specified in the Operations and Maintenance Manual. Record the voltage of the generator while lightly loaded and when fully loaded. A full-load test should be conducted every three to five years.

Full-load testing involves placing a 100 percent capacity load on a generator and allowing it to run for a while. This not only brings to light any problems in the generator and in the engine and its cooling system; it also benefits the engine by properly seating the rings and dislodging buildup in the combustion chambers and on the valves. Diesel engines especially need a periodic load test in order to maintain performance and fuel economy.

Observe the generator vibration during operation and compare the amplitude to the manufacturer’s specifications. Inspect the airflow in the generator room and verify that the generator exhaust evacuates the room quickly and safely. Verify that the airflow in the

generator area is sufficient for cooling the machinery. This can be done by feeling the machinery for excessive heat or by placing thermometers at various locations.

Pay close attention to the generator fuel tank and batteries for any leakage or corrosion. An example of an exterior grade auxiliary generator is shown in Figure 17.95.

Figure 17.95 Exterior Grade Auxiliary Generator

17.3.20 Lubrication of the Gears and Bearing Mechanisms

All gears and bearings require lubrication; however, the type of lubricant varies between different types of gear and bearings. Using the wrong type of lubrication can damage the mechanism and void any associated manufacturer's warranty. Therefore, maintenance personnel should always use the lubricant specified by the manufacturer of the gear or bearing, which should be detailed in the lubrication procedure of the operations manual. If a question arises regarding the use of a different lubricant than the one specified by the manufacturer, it is recommended that the manufacturer of the mechanical component be consulted prior to application of the lubricant. An example of a lubrication schedule and the associated details is included in Table 17.2.

While it is recommended that the lubricant specified by the manufacturer always be used, the following sections give a general overview of types of lubricants and selection criteria, in case that the manufacturer recommendations are not available.

Table 17.2 Lubrication Schedule for Exterior of Drum Girder for Swing Bridge

Item Name	Location	Drawing Number	Type of Lubricant	Application Method	Greasing 5/21/12	Greasing 6/18/12	Greasing 7/23/12	Greasing 8/20/12	Greasing 9/17/12
Right Angle Reducers	One of each side of operating machinery	10314-L3 10314-L4	Mobil GEAR 600 XP 320	Reservoir					
Right Angle Reducers Top Bearing	Top of Right Angle Reducers	10314-L3 10314-L4	SWEPCO SWEPCO 101	Grease Gun Alemite Fitting					
Right Angle Reducers Bottom Bearing	Below each Right Angle Reducers	10314-L3 10314-L4	SWEPCO SWEPCO 101	Grease Gun Alemite Fitting					
Rack Vertical Couplings (Top & Bot)	Below each Right Angle Reducers	10314-L3 10314-L4	SWEPCO SWEPCO 101	Grease Gun Alemite Fitting					
Pinion Shaft and Housing	Outside of Drum Girder	10314-L3 10314-L4	SWEPCO SWEPCO 101	Grease Gun Alemite Fitting					
Rack & Pinion	Drum Girder Rack Bottom Pinion	10314-L3 10314-L4	SWEPCO SWEPCO 164	Brush					
Slewing Bearings	48 Fittings above Drum Girder Rack	10314-L4	SWEPCO SWEPCO 101	Grease Gun Alemite Fitting					

17.3.20.1 Types of Lubrication

There are two types of lubricants for movable bridges, oils and greases. Lubricating oils are used in closed systems that typically use mechanical devices to continually lubricate gears and/or shafts. The oils can be derived from animal and vegetable products, petroleum oils, synthetic oils and solid lubricants. The fatty oils derived from animal and vegetable products tend to oxidize and develop sludge and contaminates. These oils have been largely replaced by synthetic oils that have improved material characteristics suitable for heavy machinery. Oils are generally limited to enclosed gearboxes, some radial-bearing applications and central pivot bearings on swing bridges.

In general, oils are best suited for applications where higher surface speeds are in balance with the loading conditions. Oils are used where operating temperatures are high (above 180 °F), so that heat may be carried away with system flow.

Grease on the other hand is more functional under conditions created by slower speeds and higher load relationships. Since grease has retention it has the ability to stick to interacting surfaces which also gives good protection during irregular loading (i.e., vibrations, impacting, start-ups, metal to metal contact). The downside is that greases do not function well where high operating temperatures are experienced and for the most part, are not filterable. However

they serve as excellent sealing devices themselves and in many applications do not require oil-tight sealing. Due to their simplicity they are more cost effective to design, build and operate than oil systems and usually require less frequent servicing. An example of grease on an open gear is shown in Figure 17.96.

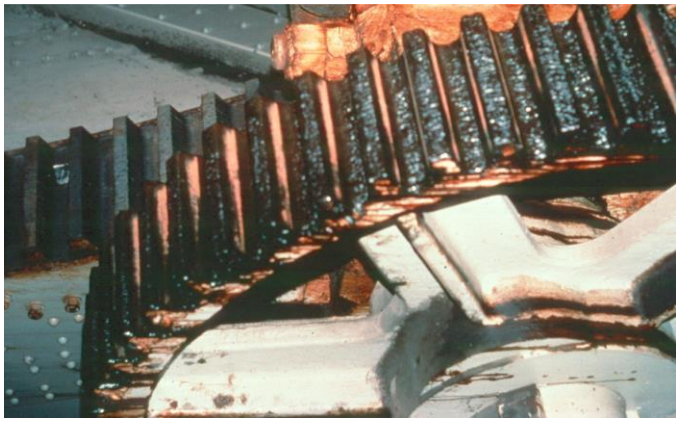


Figure 17.96 Grease on Open Gear

Greases are the preferred method of lubrication for the majority of mechanical devices due to simplicity, service requirements and relative cleanliness. Greases are essentially lubricating oils mixed with dispersions of thickening agents to create a solid or semi-solid lubricant. Greases are designed to "bleed" lubricating oil when undergoing shear. Though the thickener has lubricating properties of its own, the oil bleeding from the grease is the actual lubricating agent. As shear pressure begins to fracture the grease structure, the grease begins to bleed like a sponge being squeezed of water. As the oil film is subjected to extremely high pressures within the contact zone and for an instant undergoes a dramatic increase in viscosity. This enables the lubricant film to withstand the high contact stresses while preventing metal to metal contact between the rolling surfaces.

As a freshly lubricated roller bearing begins to rotate, the grease moves off the rolling elements and is pushed out to the housing. A small portion of it remains inside the bearing itself where shearing will take place releasing the oil lubricant. The discharged bulk of the lubricant will build wall-like damming structures along the sides of the bearing. This grease dam helps to trap the released oil product. As bearing elements continue to rotate, the grease dam will slump back in towards the rolling element slowly over time. The grease dam will then be pulled back in and behind a passing roller where the cycle will begin again. To ensure the longest possible life, it is important that roller bearings are never over filled and should be periodically cleaned out before the cavity is filled completely.

Greases are commonly used where oil circulation systems and sumps are not practical, in gearing that needs the lubricant to remain on parts or contacting surfaces, when enhanced sealing action is required due to dirty/mist environment or to protect against corrosion. On movable bridges, greases are used to lubricate a number of different types of bearings and gears. Common bearings are radial bearings, both plain and friction-less roller type, linear bearings, and thrust type bearings used on center pivoting swing bridges. Grease is used to lubricate open gears, simple spur-type gears and bevel gears because the product adheres to the contacting surfaces.

Lubricants for each component on movable bridges should be specified and followed for both repairs and maintenance actions. Lubricants are designed for the specific types of friction encountered in the mechanism. The film thickness of lubricants is affected by the speed of the machinery, the viscosity of the fluid, and the load on the machinery. Determining the proper lubricant requires a good understanding of the application, its environment, the material characteristics of the lubricants, the mechanical properties, properties of the additives, compatibility with one another, practicality, availability, and costs.

17.3.20.2 Lubrication Conditions

Three types of conditions under which the machinery is designed to function when lubricants are used to separate the machine surfaces are called hydrodynamic lubrication (commonly called full film lubrication), elastohydrodynamic lubrication (commonly called mixed film lubrication), and boundary film lubrication. Knowing which machinery operates in these environments is necessary for the selection of proper lubricants. Each and every application will produce a specific film characteristic and will require particular lubricant types and properties for a successful operation.

In full film lubrication surfaces slide on a fairly consistently thick layer of lubricant. An increase in speed or viscosity will produce a thicker film, while increasing the load will decrease the film thickness. This condition is typically achieved in internal combustion engines and is rarely, if ever, generated on movable bridge applications.

Mixed film lubrication occurs under very high pressures. Surfaces move over each other on varying layers of lubricant. On moving bridges this condition would exist in bearings on high to mid speed shafts (higher than 10 feet per minute) and between some high-speed gear engagements. Hydrostatic bearings produce low friction and have high load capacity in addition to longer life. The benefits would be ideal for the large trunnion and thrust bearings found on movable bridges; however they require a greater maintenance effort and are costly.

Boundary film lubrication conditions occur on slow moving roller and plain type bearings, including slower moving gears under extreme pressures. As the surfaces begin to rub increasingly against each other the oil film becomes thinner. Lubricant at the molecular level must be present to prevent damaging the high surface finishes. Often, additives are needed to protect the moving parts. Slow moving rotary and oscillating mechanisms are common in movable bridge applications.

17.3.20.3 Selecting a Lubricant

Selecting any kind of lubricant requires the consideration of several factors. The most important of which are the desired or anticipated mode of film condition (full, mixed, or boundary film conditions), the relative speed of moving parts, the types and size of loads, and operating temperatures of the machinery.

Misapplication of lubricants is very common on movable bridges. More lubrication is not necessarily better. Oils or greases are formulated for specific functions. Even if the correct lubricant type is specified, differences in composition by the manufacturers may result in a contaminated condition.

Some of the things to look for are: excessive oil-bleed or pooling on or around grease lubricated bearings. This may be an indication that the wrong grease product is being applied, product is

expired, bearing may be running hot, or product is contaminated. Color changes in the grease product. Darkening indicates the product has been contaminated or is oxidizing. Hardened grease is depleted of oil and not suitable as a lubricant, and should be removed.

Each of the various types of lubricants is designed for specific purposes and has characteristic properties suitable for their function. Lubrication oils and greases each have their own characteristic properties to consider.

Some of the essential material characteristics of lubrication oils for movable bridges are:

- Viscosity
- Viscosity index (VI)
- Oxidation stability
- Rust prevention
- Demulsibility or water separation
- Cloud point
- Pour point
- Lubricity
- Tackiness
- Extreme Pressure
- Consistency
- Compatibility

Viscosity is essentially the ability of the material to flow. A high viscosity indicates a resistance to flow. The *Viscosity Index (VI)* indicates the rate of change in viscosity for a specific change in temperature. A high VI indicates a minimal change in the materials ability to flow. For most mechanical operations a high VI is desirable. The most important aspect to be considered while selecting any lubricant, whether it is oil or grease, is its base-oil viscosity.

Oxidation stability measures the resistance to oxidation. Oxidation is a chemical process that will break down a lubricant. The lubricant may become acidic and form a sludge. Generally the process is slow and on regularly maintained systems it usually does not become a problem. If the lubricant is acidic, it needs to be changed immediately.

Rust prevention can be inherent in the base material or a corrosion inhibitor can be added. Good rust protection is important in applications that are open to moist environments such as trunnion-type bearings, and open gear systems.

Demulsibility (also referred to as Water-Separation) is the ability of the oil to separate from water. The number indicates the ability of an emulsion of oil and water to separate. This is an important property is applications exposed to run-off, driven rain, marine tides, and in areas of high humidity.

Cloud point is the low temperature at which the oil will begin to crystalize and become wax-like in appearance. This value is important as it indicates the temperature where the lubricant may clog filters or small orifices.

Pour point is the lowest temperature at which a lubricant can be poured from its container.

Lubricity is a measure of the film strength of the oil. Not all oils have the properties to maintain adequate lubrication where machine surfaces begin to increasingly rub against each other (boundary film conditions). Generally lubricity is enhanced with additives.

The base properties of greases are enhanced with additives. Many of the additives used in oils are common to greases and serve the same purposes. For greases, properties of tackiness, extreme pressure and consistency, and compatibility are important.

Tackiness refers to the ability to "stick" to a surface. Crater compound is commonly used, however its resistance to oxidation is very low and it does not function well in cold weather.

Extreme pressure (commonly denoted as EP) additives provide increased protection against friction and wear specifically where heavy loadings apply physical metal to metal contact (boundary film conditions).

Consistency or firmness of grease is the principal reason why grease is chosen over oil for certain applications. Consistency is the measure of the firmness of the base material. Consistency is determined by the depth at which a cone penetrates into the grease. Grease consistency may be affected by temperature cycling, time in storage, and exposure to the environment.

Compatibility is an assessment of how a particular grease will interact with other greases formulated with alternative thickeners. Non-compatible greases may solidify, loosen, and bleed-out base oils when mixed.

Thickener types are the second most important consideration in selecting grease products. Grease generally consists of a thickening agent emulsified with mineral or vegetable oil. Soaps are the most common thickening agent used, are widely produced, come in a variety of base-oil viscosities and consistencies, and can be enhanced with an assortment of additives. The selection of the type of soap is determined by the application. Soaps include calcium stearate, sodium stearate, lithium stearate, as well as mixtures of these components. Fatty acids derivatives other than stearates are also used, especially lithium 12-hydroxystearate. The nature of the soaps influences the temperature resistance (relating to the viscosity), water resistance, and chemical stability of the resulting grease.

Since grease soaps are not totally inert, they will tend to oxidize in storage and deplete themselves by bleeding-out base oil. This is the storage-ability of the product and is typically in the range of two years for most soaps. Therefore, 2 years is the absolute longest any grease product should be around, whether it's in service or not. If it is in service, the existing product should be entirely cleaned out and recharged with new grease.

Lubricant manufacturers are an excellent source of information regarding lubricant properties and additives. Industry associations such as the International Standards Organization (ISO), the American Gear Manufacturers Association (AGMA), the National Lubricating Grease Institute (NLGI), and the Society of Automotive Engineering (SAE), generally used for automobile gear oil, have instituted classification systems to aid in the selection of lubricants suitable for heavy machinery.

The NLGI consistency number (sometimes called "NLGI grade") is a measure of the relative hardness of a grease, as specified by the standard classification of lubricating grease established by the NLGI. Those with a NLGI No. of 000 to 1 are used in low viscosity applications. Examples

include enclosed gear drives operating at low speeds and open gearing. Grades 0, 1 and 2 are used in highly loaded gearing. Grades 1 through 4 are often used in rolling contact bearings. Greases with a higher number are firmer, tend to stay in place and are a good choice when leakage is a concern. Most movable bridge applications work well with NLGI 1 or 2 consistencies. Non-enclosed linear sliding bearings may consider a No. 3 grease for increased water wash stability. A lubrication expert or NLCI application guides are a good source of information for specific recommendations. NGLI consistency numbers and applications are presented in Table 17.3. Additional considerations when selecting a grease are operating temperature ranges, pump-ability, channeling, slumping, retention, and sealing ability.

Table 17.3 NLGI Classification and Consistency Numbers

NLGI number	ASTM worked (60 strokes) penetration at 25 °C *tenths of a millimeter*	Appearance	Consistency food analog
000	445-475	Fluid	Cooking oil
00	400-430	Semi-fluid	Apple sauce
0	355-385	Very soft	Brown mustard
1	310-340	Soft	Tomato paste
2	265-295	“Normal” grease	Peanut butter
3	220-250	Firm	Vegetable shortening
4	175-205	Very firm	Frozen yogurt
5	130-160	Hard	Smooth paste
6	85-115	Very hard	Cheddar cheese

Determining the correct grease for an application is made easier with the introduction of multi-service greases. With the exception of extreme conditions such as water wash, high and low temperatures, and chemical contamination, most multi-service greases, regardless of thickener type, will perform satisfactorily in the majority of applications. Other factors include, but are not limited to, the realistic time frame between service intervals, extreme temperature conditions, the presence of contamination (to include foreign matter and moisture), the possibility of metal to metal contact (impacting, vibrations, loss of lubricant), the inherent effectiveness of sealing functions, and system's cost or customer's preferences.

Any time a new lubricant is used (either a different type or the same type from a different manufacturer); all of the existing lubricant must be removed to avoid contamination. It is very important that lubricants do not become contaminated. When a lubricant becomes contaminated, it generally becomes acidic. An acidic condition over a long period of time can result in accelerated bearing and gear wear. The acidity of the lubricants should be tested annually. Removal of existing lubricant is done manually and combined with flushing with mineral spirits.

17.3.20.4 Removing Grease

An assessment of the existing grease may provide feedback as to its effectiveness. Unusual smells may indicate burnt lubricant or an oxidizing condition. Varnish or lacquer formations on bearings race are a product of an oxidizing condition. Varnish is generally associated with contamination or expired product, while lacquer is a result of an overheating condition. Small dark tarnished areas or spots on bearing surfaces may indicate formation of rust due to insufficient rust inhibiting property, product breakdown, wrong product or contamination.

One very important servicing issue is the proper method of cleaning lubricated devices. Crews should refrain from the use of solvents for cleaning purposes. Solvents are strongly discouraged since they tend to leave residue which will result in contaminated lubricant. The best method of cleaning any device is by mechanical means. Highly polished running surfaces should be cleaned using rubber gloves. If there is any residue or stiff grease that is tough to remove, try soaking parts with heated oil for a few hours or even a day if need be. On parts that still present a problem, kerosene is very effective in loosening thick black clogged product. Afterwards, flush continuously with light mineral oil before relubricating.

Sometimes grease deposits are so hard and thick they look like plastic. In this case, solvents could be considered. Once the fouled product has been removed allow the solvent to evaporate, hair dryers work perfectly. Then flush continuously with light mineral oil until all solvent residue has been removed and re-lubricate. It is imperative that no solvent is left to contaminate fresh lubricant. If unusual bleeding occurs shortly after a solvent was used in the cleaning process, it may indicate that the new grease has been contaminated.

Most lubrication problems are easily resolved with minimal effort. On the other hand, some problems can be much more complicated and may require an extensive effort. Part of exercising a good preventive maintenance effort is staying alert and paying attention to the slightest details of each component. Keeping things clean and monitoring each device as frequent as possible during operation is a good practice. An excellent way to monitor a device is to lay a hand on it during operation. You will become accustomed to the way it heats and vibrates during operation enabling you to detect any early signs of problems.

Lubrication misuse is perhaps one of the greatest causes of long term mechanical failures. Application of lubricants needs to be careful and consistent. Often times a lubricant is specified for use on sleeve type bearings such as EP-2 grease. Often those using the product do not understand that EP lubricants tend to become corrosive when exposed to contaminants, moisture and temperature. If these lubricants are not purged through the bearing quickly enough, their chemical reactions can cause degradation in the bearing journal. This effect is commonly seen on counterweight trunnion shafts that look like they have been scored by some foreign material. This scoring usually occurs due to chemical reaction with the lubricant accelerating the phenomenon of “micro-pitting”, as described below.

Micro-pitting is where the bearing surfaces contact at the microscopic level resulting in extremely high pressure, such that the material actually flashes due to the heat. This situation occurs over and over again until such time as the shaft appears to be scored and the trunnion friction increases.

The other common problem with lubricants is that when they chemically react some types become hard, preventing new lubricant from reaching the surfaces under load. The result is

that these surfaces are in effect, not lubricated. One of the most common causes of lubricant contamination is the addition of a different type of lubricant. Unfortunately, not all lubricants are compatible with one another. If changing brands of lubricants, it is likely the new lubricant will react adversely to the existing lubricant.

When removing existing lubricant from closed mechanical components such as reducers and trunnion, inspect the grease for the presence of metal particles. If metal particles are observed in the grease, this is a sign that the gears are being worn and an internal inspection of mechanical component is warranted.

What To Look For

Metal particles in the grease are an indication of worn gears.

Figure 17.97 shows grease from a trunnion bearing on a bascule bridge that exhibits bronze particle pickup.

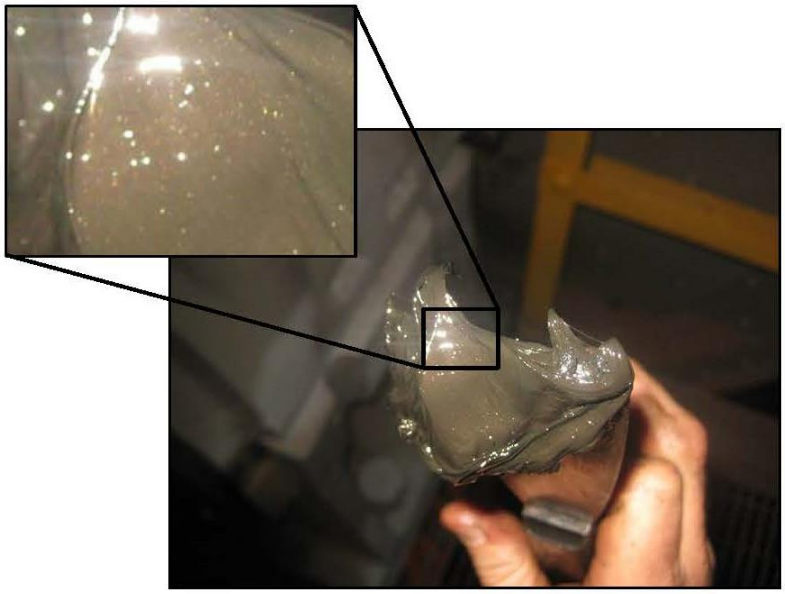


Figure 17.97 Grease Exhibiting Extensive Bronze Pickup

17.3.20.5 Lubricating Movable Bridge Components

The two most commonly recurring types of damage on the components of movable bridges are leakage from the gear box and inadequate or improper lubrication of the gearing. Over lubrication can also have detrimental effects on the machinery components and may lead to the failure of internal seals on bearings and couplings, an example of which is shown in Figure 17.98. Early detection of these faults may help to avoid operational problems and major damage to the machinery.

Figure 17.98 Trunnion Bearing with Broken Seal Due to Excessive Lubrication

Open Gears

Open gears are susceptible to corrosion due to a lack of lubrication, excessive strain, out-of-plane rotation and misalignment. If the gear teeth are not kept lubricated at all times, wear and corrosion due to grinding will occur. An example of a properly lubricated open gear is shown in Figure 17.99.

The partial rotation of the rack and pinion is done at low speed, then reversing direction results in metal-to-metal contact occurring within the bearing. These types of bearings are typically lubricated with an Extreme Pressure (EP) grease to aid in supporting the load. Solid molybdenum disulfide (moly) or other additives are added to the grease to enhance the load-carrying capability of the lubricant.

Generally, if the open gear lubricant is to be applied by a drip system, force-feed lubricator or spray system, it must be sufficiently fluid to flow through the application equipment. For brush applications, the open gear lubricant must be fluid enough to be brushed evenly on the teeth. In any case, during operation, the open gear lubricant must be viscous and tacky enough to resist squeeze-out from the gear teeth. When open gears are lubricated by dripping into a splash pan or through the use of splash and idler immersion systems, the open gear lubricant must not be so heavy that it channels as the gear teeth dip into it. Finally, when open gears are lubricated, the consistency or grade and its ease of pumpability must permit easy application under the prevailing ambient conditions.

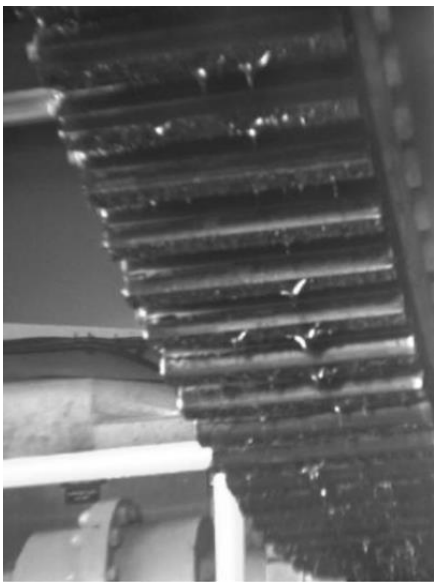


Figure 17.99 Properly Lubricated Open Gear

Closed Gears (Speed Reducers)

Access to the closed gear sets is generally by ports (or covers) which can be removed. It is quite difficult to measure the backlash of the gear sets in an enclosed reducer.

Bearings

Greases are used to lubricate most of bearing types found on movable bridges. However, some bearings and thrust bearings on swing bridges are lubricated with oils. For oil lubricated bearings, the oil level should be just high enough to submerge half of the lowest bearing to prevent churning and foaming. The use of EP additives should be avoided. Oils that do not contain solid lubricant additives should be used, as solid lubricants work as a contaminant in this application.

Thrust bearings commonly experience extremely slow speeds and are required to support millions of pounds (boundary film condition). The proper oil lubricants for thrust bearings should have a high viscosity, but not excessively high to where it will restrict good penetration into the contact area. The selected oil should also have excellent lubricity, good rust and oxidation stability, and be EP rated. Solid lubricant additives should also be considered.

Thrust bearings on swing bridges cannot be accessed for cleaning and greasing. Therefore, oil bath systems are normally used. The oil level is maintained at some point above the highest elevation of the upper disc so that the entire interface is submerged. Oil is usually drained from center of the lower bearing. Oil grooves are provided in the bronze disc to provide lubricant coverage, which also facilitates settling out of contaminants into the lowest point.

Sleeve bearings require lubrication between the sliding surfaces of the journal and bushing. It is important that the bearing be kept full of fresh clean grease and be periodically cleaned of old grease. Normally one or more grease fittings are located at the top of the housing. When lubricating the bearing, it should be confirmed that old grease exits from the space between the journal and bushing. The disadvantage of these bearings is that they cannot be conveniently disassembled for cleaning and have the tendency to retain old product despite the best re-lubrication efforts. Old grease will eventually breakdown and may become reactive and oxidize, solidify, bleed out, and/or produce lacquer or varnish type coatings on journal surfaces. These

coatings are destructive in that they may prevent certain additives from developing oil-soluble surface films.

The sleeve bearings should be well secured and free of damage. The bearing housing should be free from cracks. There should be minimal radial movement during operation to minimize wear on the bushing. The clearance between the shaft and the bushing should be measured, using feeler gages, and the measurement recorded. The exterior housing of the bearing should remain cool to the touch. Any heat generation may indicate improper lubrication, or damage to the bearing.

The best grease product for sleeve bearings are those with a high base-oil viscosity, are extreme pressure (EP) rated, and contain solid lubricant additives for added protection. In addition these products should have good oxidation and corrosion protection for increased service life.

Selecting grease products for roller bearings should have a NLCI consistency rating that will provide good channeling as well as be able to slump throughout the entire operating temperature range. Base oil viscosity needs to be just heavy enough to develop film integrity, yet light enough to prevent adverse temperature conditions and churning.

Linear bearings operate under slow speeds and high loads (boundary-type film conditions). Lubrication that can withstand extreme pressures in the bearing and the ability to withstand direct exposure to rain or tidal action, water run-off, and/or scraping-off must be extremely tenacious in nature. Crater grease, sometimes called grease tar, is a residual asphaltic compound well suited for this application. These compounds are usually brushed or sprayed on with solvents; pay attention to health warnings.

Some linear bearings are designed to operate using more standard type grease products. In these applications, a NLCI No. 2, 3, or even 4 EP rated greases with solid lubricants, good channeling, minimal slumping, tackiness agents, water stable, that are oxidation and rust stable are best suited. Due to the harsh environment, frequent servicing is advised for optimum life.

Bearings supporting the shafts in the reducer are generally mounted in the housing and are not readily accessible. After the unit has been operated, these bearings can be touched to determine if they are getting warm. If a bearing becomes warm after one or two bridge openings, it should be checked.

Wire Ropes

A wire rope is constructed by spinning strands of wire into a bundle. These bundles are then spun around a core to construct the rope. The typical ropes are 6 by 19 with a fiber core. The 6 indicates that six (6) bundles of nineteen (19) wires are wrapped around the core. As the group of bundled wire passes over drums or run over sheaves, friction between the bundles and the wires will occur. To minimize abrasion within the rope, the ropes must be properly lubricated. If a rust color appears on the rope it should be carefully inspected. It is important to understand that wire ropes have a tendency to fail from the inside out. Wire ropes which are properly installed will provide decades of reliable service. An example of wire rope sockets and fittings is shown in Figure 17.100.

Figure 17.100 Wire Rope Sockets and Fittings at Lifting Girder

Wire ropes are used for counterweights, auxiliary counterweights and for operating the span (span drive vertical lift bridges). In general the maintenance required for wire ropes is to assure that they are properly lubricated, that the wire rope tension is correctly set, and that the sheaves that the ropes run over are functioning properly. An example of wire rope lubrication is shown in Figure 17.101. Wire ropes should be visually checked once a year for broken wires, rusting, evidence of rubbing during operation, and flat spots. For an example of fouled wire ropes, see the photo in Figure 17.102. Flat spots greater than 1/4 inch long for ropes under 1-inch diameter and greater than 1/2 inch long for ropes over 1-inch diameter should be brought to the attention of the engineering staff.

When to Call the Engineer

If flat spots are discovered on wire ropes which are more than ¼ inch long on ropes less than 1 inch diameter, or more than ½ inch long on ropes over 1 inch diameter.

Figure 17.101 Lubricating Wire Ropes (Courtesy of California Department of Transportation)

Figure 17.102 Fouled Wire Ropes (Courtesy of California Department of Transportation)

The largest components on a vertical lift bridge are the counterweight sheaves. From engineering and design criteria, it is imperative that the wire rope grooves, the pitch line, and the trunnion journals are all concentric to evenly load all ropes and operate smoothly.

When replacing counterweight ropes, wire rope shrinkage will occur and the counterweight wire ropes will not regain their preloaded length until the ropes are actually operated. After installing a new rope, it is necessary to run it through its operating cycle several times (known as break in period) under light load (approximately 10 percent of the Working Load Limit) and at reduced speed. Start with light loads and increase gradually to full capacity. This allows the rope to adjust itself to the working conditions and enable all strands and wires to become seated. Depending on rope type and construction, some rope stretch and a slight reduction in rope diameter will occur as the strands and core are compacted. The initial stretch (constructional stretch) is a permanent elongation that takes place due to slight lengthening of the rope lay and associated slight decrease in rope diameter. Constructional stretch generally takes place during the first 10 to 20 lifts.

Wire Ropes are usually made slightly larger than nominal diameter to allow for reduction in size which takes place due to the compacting of the structure under load (break in period). Keep a record of the new rope diameter after break in period for future reference.

The drums and sheaves have grooves which control and support the wire ropes. If slippage between the rope and the grooves occurs, then both the rope and groove wear. This wear is accelerated when a sheave bearing fails and the rope is dragged across the sheave. The best way to identify these problems is to observe the operation of each sheave while the span is in operation. Unfortunately no adjustments are available for the rope groove size. When rope grooves become too large, the wire rope deforms from a round to an oval shape as it passes over the drum or sheave. This deformation accelerates wear of the ropes.

17.3.21 Alignment

Seasonal and daily temperature changes, as well as sudden temperature shocks, affect the alignment and fittings on different components of the bridge. As the bridge materials expand or contract due to temperature changes, alignment of the mechanical components can be affected. This may lead to an increase in friction between moving parts. During bridge

operation, significant changes in mechanical friction may be an indicator of a need for maintenance or adjustments in component alignment. For example, in summer, the operators on the West Third Street Bridge in Cleveland, OH are often seen watering down the end, not to keep the dust down, but to keep the steel cool so the bridge does not bind.

Live load bearings, when misaligned or improperly balanced so that the bridge is not fully seated, cause the dead load and traffic load (live load) to be transferred to the gears and shafts. This puts an enormous strain on the machinery. Small gaps also lead to the girders pounding on the live load bearings, which results in further misalignment, additional stresses, fatigue damage and excessive wear.

The operation of the live load bearings should be checked to assure consistent proper seating. A simple method to assure consistent operation is to place indicators on the bearings and the seat (e.g., using marking crayons) to check consistency of operation. As bearings become worn or improperly adjusted, they can become “stuck”. Sometimes the lubricant has chemically reacted causing the mating surfaces to become rough (increased coefficient of friction). Sometimes the drive linkages are so worn that a significant amount of power is lost to machinery inefficiency (the drive system does not have sufficient power to pull the bearings under extreme condition).

Span lock sockets are typically welded or cast housings equipped with adjustable wear shoes. Adjustment is made by adjusting the shim material. The lock bar is operated within the sockets by a crank mechanism, which is driven by gearing and a motor. Linear lock bar operators are also common. The locks should be adequately lubricated. There should be little, if any, relative span movement of the locks under traffic. If movement is observed, gaps between bar and sockets should be measured with feeler gages and recorded.

The alignment of the guide socket and receiving sockets and lock bars should be match marked, aligned and shimmed at manufacturing point duplicating field conditions as closely as possible. This may require re-aligning when actually assembled into place. Any locking system should be properly aligned and fastened rigidly to prevent any possible movement. If movement does take place, the locking systems are easily misaligned and will not realign themselves. The locking system is not a bridge alignment tool and if not properly aligned damage will occur.

Some common problems are associated with locking systems, alignment, air buffers, shop tolerances versus field practices, and translation of design drawings into workable shop detail drawings. From a practical standpoint, it is advantageous to pre-assemble and simulate the field conditions as much as possible in the shop, to eliminate as many field problems as possible and to insure proper operation in the field.

17.3.21.1 Repair of the Pivot/Rack Assembly

The rolling bascule bridge is a straight forward manufactured mechanical system with a straight rack and pinion, a machinery frame and the tread and track plates. The tread and track plates may be fabricated out of high strength, low alloy plate steel, machined, and have the lugs either pressed in place or machined.

The trunnion type bascule bridges rotate around their trunnions and trunnion bearings and are commonly driven through curved racks that are bolted or fabricated to the rack support. The movement is designed to follow the pitch line (or pitch circle). The pitch line usually cuts the

teeth at about the middle of their height. When aligned properly, movement occurs without backlash (loose play in the gears).

Shims can be used to eliminate an inconsistent backlash with the use of shims placed between the rack support and the girder. It is important that the pitch diameter be consistent with the top flange of the rack support. In addition, for swing spans, the track must be installed level and in true relationship to the center pivot to establish proper tracking with the balance wheel assemblies.

Repairs to the track and tread should be done in the shop, when possible. The track and tread should be actually blued, (a chemical process that provides some corrosion protection) match marked and rolled the length of travel and tested after all machine work is completed to insure proper contact and tracking ability. A track example is shown in Figure 17.103.

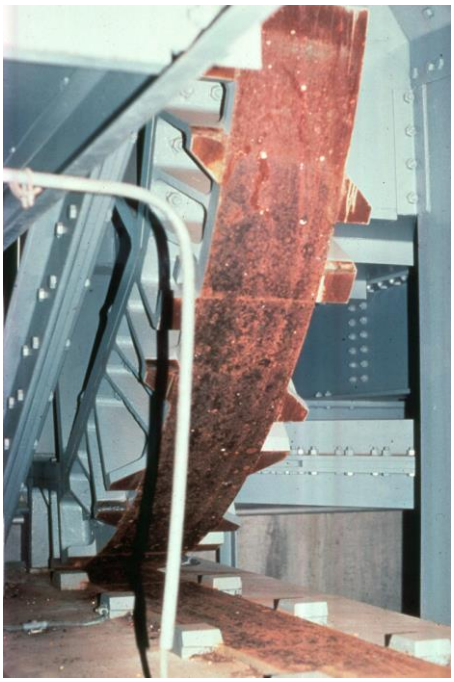


Figure 17.103 Track (Courtesy of Wisconsin DOT)

Extremely accurate measuring equipment must be available during the repair. Sometimes, specialty tools may be required that may have been supplied by the mechanical supplier after the initial installation.

Decisions regarding the operation system and mechanical parts require special expertise and knowledge of the structure. A malfunction of any component can cause an unexpected failure of bridge operation.

17.3.21.2 Balance Testing

Balance and alignment are affected by rehabilitation, repairs and deterioration. To track bridge balance and alignment and other effects on the operation, such as environmental effects, the bridge balance should be monitored and performed real-time for each bridge operation. Incorrect balance condition will lead to different support reactions and internal stresses and a fundamentally different behavior. Measurements taken as the bridge is lifted and closed are used to determine whether or not the bridge requires balancing.

Bridge balancing and shimming is necessary on all movable bridges as they deteriorate. The balancing procedure is relatively unsophisticated, relying on current draw from the motors to gage relative balance. Engineers oversee the balancing operation to determine whether or not to add or remove counterweight blocks to achieve balance. Bridge shimming is critical for proper seating of the bridge when closed and under load. Bascule bridges are shimmed to within approximately one-quarter inch of full bearing on the anchor column and "just seated" on the live load bearings under a "no live load condition”.

Swing bridges may be symmetrical or to have unequal length arms. Arms that are not of equal length are termed unsymmetrical or bobtailed. The dead load (self-weight) of a swing span is usually balanced about the pivot. Hence, bobtailed spans require counterweights at the ends of the shorter arms. When open, the swing bridges must maintain their own weigh. When closed, the spans are supported by a center pier and abutment at each end with wedges driven in to lift the bridge ends, preventing damage to the bridge under live load conditions. As a result, swing bridges are generally either center-bearing structures or rim-bearing ones.

The bridge load in center-bearing swing bridges is carried on the central pivot with balance wheels placed on a circular track on the outside of the central pivot to prevent tipping. Balance wheels are found only in center-bearing swing bridges. As the structure load is balanced on the center pier, it is vital to have the mechanism support unbalanced loads. In rim-bearing swing bridges, the pivot pier places the structure load on a circular girder or drum to beveled rollers. These rollers are located within the pier on a circular track, spaced by concentric spacer rings . The central pivot in this type of swing bridge also carries part of the structure load.

There are three main ways to perform a balance test:

- Drift Test
- Strain Gauge Testing
- Motor Current Balance Testing

The most accurate methods of balance testing involves the use of strain gauges and electrical current tests.

For bascule and lift bridges, drift tests can be used to make correcting adjustments to the counterweight. To conduct a drift test, the span is raised to 10 degrees open and held. The brakes are then manually released. The spans should slowly drift to the closed position (be sure the brakes are working properly before proceeding). The test is repeated at 10 degree intervals until the spans are fully open.

If properly balanced, the span should drift to close when held at the near half open positions. Near the mid-open point the spans may either drift closed, remain in the mid-open position, or drift open. If the span drifts down too quickly or drifts down from the full open position, it may be too span heavy. Spans that don't drift down are counterweight heavy. Counterweight adjustments are made accordingly to maintain the span in proper balance.

Strain gauge testing incorporates strain gauges to measure the torque in the span drive pinion shaft. This method can be used on spans that are not driven by rack and pinion drive systems, such as rack and pinion bascule spans.

Motor current (or amperage) balance tests use ohm meters to determine the current draw on the electrical motors that control the movement of the spans. Variations in the current drawn by the motors provides an indication that the span may be out of balance.

These tests should be performed under the direction of an engineer. Based on information from balance testing, recommendations can be made concerning balancing and/or lubrication of components.

When to Call the Engineer

Call the Engineer for direction in performing balance testing.

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Chapter 19 - Maintenance and Preservation of Ancillary Features

19.1 Utilities

19.1.1 Record of Utility Installation History

Bridge maintenance crews are not typically responsible for the maintenance of utilities on bridges, but will need to govern their work to avoid disrupting them. Bridge maintenance operations requiring interruption to utility services should be coordinated with the utility operator. Bridge maintenance crews should also check the condition of utilities mounted on the bridge for any obvious signs of deterioration, notify the utility owner if deterioration is identified and be familiar with any agreements between the bridge and utility owners.

New utility installation or re-configurations should be reviewed and/or field verified prior to any new documentation entry. Utility attachments should not be allowed if the bridge load rating would be reduced below the legal limit. Any requests, or actions, to add or modify utilities should be coordinated with the bridge office. Methods of attachments and any additional loads due to the utilities should be reviewed by the bridge office.

Bridge utility occupancies should be identified and recorded on a “Bridge Information Data Sheet”. The current “What, Where, and How” information should be documented, as shown in the example data sheet in Figure 19.1. It can be beneficial for bridge owners to provide coordination between the Bridge Management System and Bridge Maintenance Management Systems so that maintenance workers and inspectors have the most up-to-date information.

BRIDGE DATA SHEET
(FOR ALL STRUCTURES)

Prepared By: Date:

DESCRIPTION

1. Bridge Identification Number (BIN)
2. County:
3. Town:
4. City/Village

GENERAL

5. Over Roadway Description
   a. State/County/Local Highway Name
   b. Route/Road Number
   c. Functional Classification NHS System
   d. Design Speed
   e. Truck Traffic %
   f. Traffic Year
   g. Traffic Data/ AADT
   h. Seismic Performance Criteria
6. Under Roadway Description
   a. State/County/Local Highway Name
   b. Rout/Road Number
   c. Functional Classification NHS System
   d. Design Speed
   e. Truck Traffic %
   f. Traffic year
   g. Traffic Data/ AADT
   h. Minimum Vertical Clearance
   i. Minimum Horizontal Clearance
7. Maintenance of Existing Traffic
   a. Is there vehicular traffic which must be maintained?
   b. If yes, how will it be maintained?
   c. If a temporary detour structure is required, what liveloading should be used?
   d. Does pedestrian traffic need to be maintained?
   e. If yes, how will it be maintained?
   f. Is the bridge part of a designated bicycle route?

EXISTING STRUCTURE

8. Structure Description:
9. Original Construction Contract No.(s)
10. Type of structure, span, and skew
11. Are plans available? Do they accompany submittal?
12. Is there any asbestos on the existing bridge?
13. Is the bridge within or adjacent to an area of contaminated material
14. Are there sidewalks on the bridge
15. Specify any utilities carried on structure
    a. Do any of these utilities need to be kept in service at all times?
    b. Can any of these utilities be interrupted for a given period of time?

Figure 19.1 Example Bridge Data Sheet

19.1.2 Utility Ownership Identification

A wide variety of utility occupancies may be present on or around a bridge structure. The utility type, location, approximate location and contact information for the utility department responsible for the carried facility should be documented. This information will assist in notifying the responsible party when problems or deficiencies are encountered with or affecting a utility facility or its support structure.

19.1.3 Utility Types

Many types of utility facilities may exist on, around, or under a bridge structure. Communication lines for telephone, cable TV, or highly delicate fiber optic cables may be visibly present or encased in a protective conduit, as shown in Figure 19.2 through Figure 19.4.

Figure 19.2 Visible Electric Lines Above and Under a Bridge Structure

Figure 19.3 Communication Lines Encased in Protective Conduit

Figure 19.4 Water Line Crossing Mounted to Bridge

Overhead lines may be present near the bridge structure, as shown in Figure 19.5. High voltage electric lines may need to be de-energized prior to the operation of cranes and booms within prescribed distances from the facility. Overhead lines may approach a bridge structure and then transition underground with the use of a riser assembly located on adjacent poles, an example of which is shown in Figure 19.6. Street lighting, traffic signal conductors, ITS sensors and similar systems owned by the transportation agency may also be attached to the structure. An example of bridge lighting attached to a bridge structure is shown in Figure 19.7.

Figure 19.5 Example of Overhead Lines Near Bridge Structures

Figure 19.6 Examples where Overhead Lines Transition Underground

Figure 19.7 Example of Street Lighting Attached to Bridge Structure

Many times an older structure may have electric or telecommunication utilities provided in tiles in sidewalk. Before any bridge rehab work is done, crews need to verify or review the bridge plans to make sure there is no conflict.

Fluids, such as sewer and high pressure water lines, and flammables, such as natural gas or oil transmission lines, may be attached to the top, fascia, or underside of the structure. Figure 19.8 shows examples of fluids which are (a) attached to the fascia overhang, (b) suspended from diaphragms, and (c) mounted to bridge fascia.

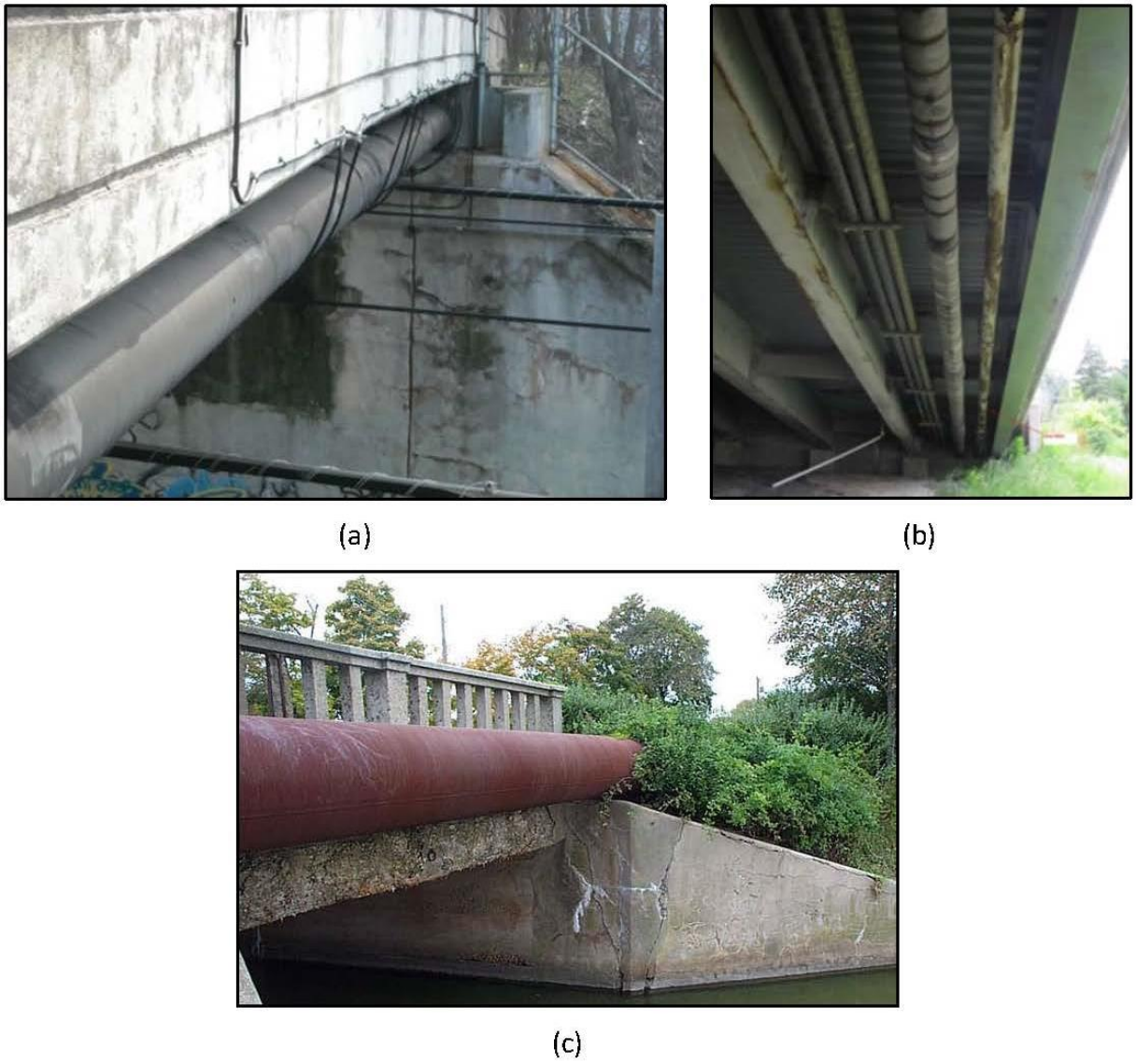


Figure 19.8 Examples of Fluid and Flammable Lines

Signage or stamped lettering may detail the type of facility and/or its contents to make all aware of the potential dangers connected with the utility encountered. Examples of signs designating potential dangers are shown in Figure 19.9. Some hazards may not be clearly labeled, but should be communicated to personnel working with or near the carried facility.

- Older conduits may consist of asbestos reinforced cement, or “transite”. Cutting, breaking and machining asbestos-containing transite elements releases carcinogenic asbestos fibers into the air.
- Energized utilities can present an electrocution hazard, either through direct contact or stray voltage.

Figure 19.9 Various Signs Designating Potential Dangers

19.1.4 Utility Attachments

Utility facilities may be attached to a bridge structure in a variety of ways. Mounting hardware such as straps, bands, or hangers may be utilized to attach the carried facilities to bridge rails, parapets, abutments, sidewalks, structural slabs, or bridge superstructure components. Examples of utility hangers on a bridge structure are shown in Figure 19.10. Utility lines are generally run through a conduit or protective casing which is attached via an accessible connection system, as shown in Figure 19.11. The utility casing or mounting hardware should not present a hazard to the public or be detrimental to the bridge structure condition.

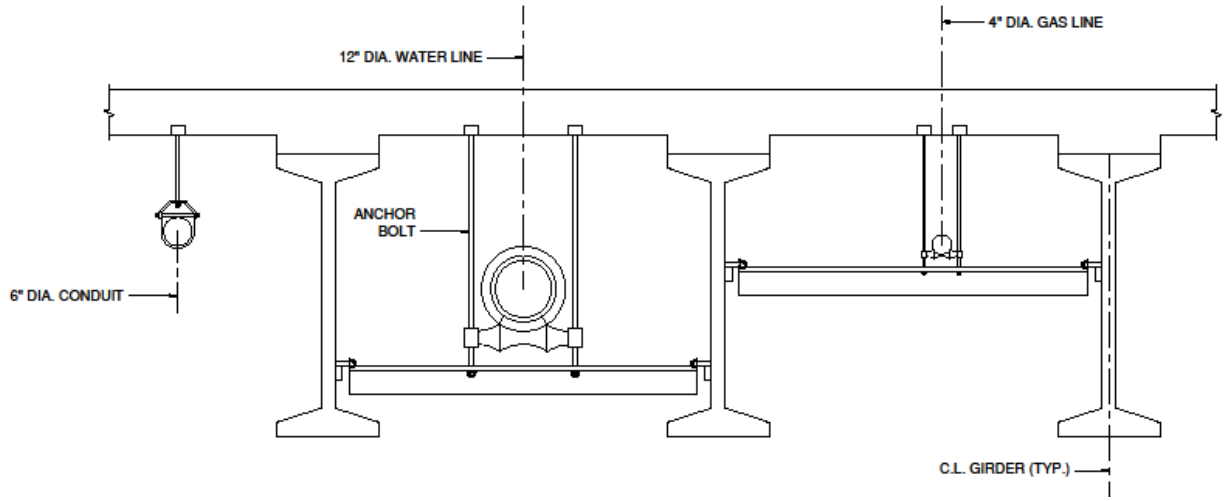


Figure 19.10 Utility Hangers on Bridge Structure

Figure 19.11 Utility Lines Running through Conduit (left) or Protective Casing (right)

19.1.5 General Maintenance Activities and Responsibility

Items to look for include:

What To Look For

- Deficiencies with the mounting hardware
- Breaks or cracks in conduit casing and connection joints
- Abnormal wear or deterioration
- Evidence of rust or staining
- Expansion or contraction problems
- Collision damage to the utility or support system
- Any damage to the bridge components

During routine bridge inspections things to look for and note are deficiencies with the mounting hardware, breaks or cracks in conduit casing and connection joints, abnormal wear or deterioration, evidence of rust or staining, expansion or contraction problems, collision damage to the utility or support system and any damage to the bridge components. Bridge Inspectors should be inspecting the utility carrier members and the support system, and should note any problems encountered with the bridge structure connections and/or its functionality. Note that the maintenance worker should also be on the lookout for these situations.

For gas mains, it should be noted that if the inspector, contractor, or maintenance worker encounters any of the following issues: natural gas odor, obvious corrosion to exposed gas pipe, or mechanical/structural damage to the gas main facility, they should immediately notify the gas company involved and call for an on-site inspection. Examples of corroded gas pipes are shown in Figure 19.12.

Figure 19.12 Examples of Corrosion at Exposed Gas Pipe

A check for water leakage from the bridge structure, either directly or in-directly, should be included to protect the utility from damage. If leakage is detected, it should be corrected by the bridge owner. If water penetration of a utility facility is evident, it should be corrected by the utility owner. An example of water leakage under a bridge structure is shown in Figure 19.13.

Figure 19.13 Water Leakage under Bridge Structure

For electrical or communication lines, a check for poor insulation coverage, stray current or loose wires if visible, is prudent to prevent potential shock events. Bridge movement due to operational vibrations, or environmental effects such as wind, temperature change, settlement, etc., should be considered in a utility placement. Earthquakes can cause sudden and sometimes drastic movements to a bridge structure. The utility facility and mounting system should be designed to be flexible enough to withstand these movements and displacements resulting from both normal operation and extreme events. An example of signs of damaged that need to be fixed is shown in Figure 19.14.

Figure 19.14 Signs of Damage Should Be Fixed to Prevent Electrical Shock

The utility facility and its support system should operate in harmony with bridge expansion devices or have a support system that acts independently of bridge movement and can facilitate the anticipated movement. Periodic monitoring with routine inspections is recommended to assure that movement events are being accommodated. Bridge Inspectors should be inspecting the utility carrier members and the support system, and should note any problems encountered with the bridge structure connections and/or its functionality. Again, the maintenance worker (in addition to the inspector) should be on the look-out for these situations.

19.1.6 Maintenance Operations and Affected Utilities

When maintenance activities or necessary bridge work will be in close proximity to a utility facility, a "Utility Protection Call Center" should be contacted and the utility owner notified by the project manager or field foreman. An example of a "Utility Protection Call Center" sign is shown in Figure 19.15.

Figure 19.15 An Example of a "Utility Protection Call Center" Sign

The utility owner should be advised on the necessary work and have the appropriate location mark-out identifications completed prior to the start of work. This notification should occur at least one week prior to the commencement of work, but not beyond two weeks of lead time, due to the potential changes that may occur through the utility owner's work that may be planned in the work area. Uniform Color Codes for Temporary Marking of Underground Facilities have been established by the American Public Works Association (APWA). The color codes are shown in Table 19.1.

Table 19.1 AWPA Uniform Color Codes for Temporary Marking of Underground Utilities

Color Code	Underground Utility
Red	Electric lines, cables, conduit, and lighting cables
Orange	Telecommunication, alarm or signal lines, cables, or conduit
Yellow	Natural gas, oil, steam, petroleum, or other gaseous or flammable material
Green	Sewers and drain lines
Blue	Drinking water
Purple	Reclaimed water, irrigation, and slurry lines
Pink	Temporary survey markings, unknown/unidentified facilities
White	Proposed excavation limits or route

19.1.7 Broken Utility Components and Responsibility

If broken supports, attachments, mounting hardware, damaged conduit or facility apparatus are encountered by either inspectors or maintenance workers, the utility owner should be notified and repairs promptly made by the utility owner representative. An example of damaged supports that require attention is shown in Figure 19.16.

Figure 19.16 Example of Damaged Supports Requiring Attention

Cracked, splintered, or severely checked wood poles and supports, such as those shown in Figure 19.17, should be noted and repaired or replaced by the owner as soon as practical.

Figure 19.17 Examples of Cracked and Splintered Wood Poles (left) and Supports (right)

If the bridge owner causes damage to a utility component or attachment while performing routine maintenance work, it is the bridge owner who is responsible to repair the damage at the direction and guidance of the utility owner representative. An example of such damage and the ensuing repair are shown below in Figure 19.18.

Figure 19.18 Example of Incidental Damage (left) and Resultant Repair (right)

19.2 Bridge Mounted Sign Structures

This section provides a brief coverage of maintenance activities related to bridge mounted sign structures. For readers responsible for the maintenance of sign structures and other ancillary highway structures, more comprehensive coverage can be found in the FHWA publication "Guidelines for Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaries, and Traffic Signals". This report can be found at the following FHWA web link: http://www.fhwa.dot.gov/bridge/signinspectiontoc.cfm.

19.2.1 Record of Sign Inventory

FHWA strongly recommends (but does not currently mandate) that agencies responsible for sign structure maintenance maintain an inventory of overhead and bridge mounted structures and perform regular inspections. Sign inventory data is a collection of structure specific information, maintained for the benefit of the owner's planning, maintenance and operational endeavors related to each sign structure. Sign structures should be uniquely numbered for easy field identification and record keeping. Numbers should be assigned during design, fabrication, or installation and should be stenciled, etched, or printed on the sign structure in clear view, as shown in Figure 19.19.

Figure 19.19 Example of Sign Structure Identification

In order for the collected data to be searchable and sortable; entry is preferably entered through drop-down menus.

19.2.2 Collected Inventory Information

Historical data such as record plans or as-built drawings are typically difficult to obtain for bridge mounted sign structures. For this reason collection of inventory data is critical to the management of the structures. A wide variety of sign data may be collected and maintained in a sign structure inventory database, depending on the needs of the database users (maintenance forces, highway designers, traffic safety groups, oversized load permitting, inspection, etc.). Some data points that may be of interest to a maintaining agency would include:

- Route number and direction
- Milepost location
- Political Unit (County/Town)
- Maintenance Jurisdiction

- GPS coordinates
- Erection Date and Contract #
- Material (Aluminum/Galvanized Steel/Weatherized Steel)
- Sign text & reflectivity
- Sign panel dimensions and location
- Inspection frequency
- Inspection date
- Inspection element ratings
- Significant findings and repair recommendations
- History of repairs
- Horizontal and vertical clearances
- Geometry data

Elements on sign structures can be highly repetitive. To aid in the tracking and locating of defects between inspection and maintenance, a common naming and numbering convention should be employed.

19.2.3 Database Inventory Information

The collected bridge sign data should be made available to all interested parties. The data should be presented in a simple format that all can utilize to sort, prioritize, or categorize as needed for the management and the continued safe operation or eventual removal/replacement of the structure.

19.2.4 Sign Inspections

FHWA does not currently mandate inspection of sign structures, however many jurisdictions follow FHWA's recommendations regarding sign structure inspections as described in publication FHWA NHI 05-036, *Guidelines for the Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaires and Traffic Signals*. If possible, bridge mounted sign structures should be inspected in conjunction with the bridge on which they are mounted.

Sign inspections can be broken down into five categories:

1. Initial Inspection – a "hands-on" inspection which takes place shortly after installation to check fabrication and erection and to collect inventory information.
2. Routine Inspection - Simple ground level inspection on an assigned frequency or after a potentially damaging event to view overall condition and screen for deficiencies or structural deterioration requiring a more detailed inspection. Visual aids such as binoculars may be used to improve coverage. Some inventory data may be collected or updated during a routine inspection. This level of inspection may be scheduled at the beginning of sign structure assessment program or immediately after a regional high-wind event.
3. In-Depth Inspection - a "hands-on" inspection activity performed at regular intervals to inspect the structural and safety element and to collect/update inventory information. All elements including bolts and welds are visually and tactically inspected.

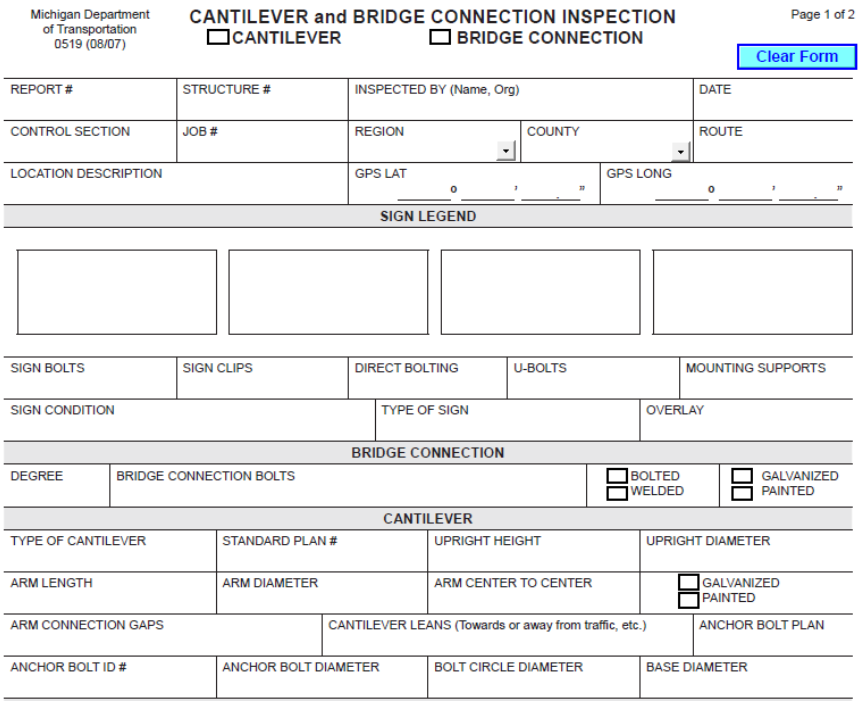
4. Interim Inspection – a "hands-on" inspection activity focused on tracking previously identified critical defects or deficiencies between the regularly scheduled In-Depth inspection intervals.
5. Damage Inspection – a "hands-on" inspection activity scheduled shortly after vehicular impact to visually and tactically inspect all elements along the impact's load path that may be affected by the event.

The frequency of inspections is determined by the responsible maintenance agency and should consider several factors, including:

- Material type
- Structural redundancy
- Bridge inspection schedule
- Traffic control needs
- Historical performance of similar structures in the inventory

Inspection reports should describe the observed deficiencies on all critical elements, where a critical element is defined as a portion of the structure that alone can affect the stability or capacity of the structure. All significant deficiencies should be noted, located, photographed and described in a manner that can easily be found by maintenance forces or used for comparison by future inspectors. A formal numerical scale, defined by the owner, is typically used to rate and track each structural and safety element.

Figure 19.20 below shows an example form that could be used to collect bridge mounted sign inspection information. Information to be collected includes how the sign is mounted to the bridge, type of sign, sign legend, and inventory information.



Michigan Department of Transportation 0519 (08/07)

CANTILEVER and BRIDGE CONNECTION INSPECTION

☐ CANTILEVER ☐ BRIDGE CONNECTION

Page 1 of 2

Clear Form

REPORT #	STRUCTURE #	INSPECTED BY (Name, Org)		DATE
CONTROL SECTION	JOB #	REGION	COUNTY	ROUTE
LOCATION DESCRIPTION		GPS LAT ° ' . "	GPS LONG ° ' . "	

SIGN LEGEND

SIGN BOLTS	SIGN CLIPS	DIRECT BOLTING	U-BOLTS	MOUNTING SUPPORTS
SIGN CONDITION		TYPE OF SIGN		OVERLAY

BRIDGE CONNECTION

DEGREE	BRIDGE CONNECTION BOLTS	☐ BOLTED ☐ WELDED	☐ GALVANIZED ☐ PAINTED

CANTILEVER

TYPE OF CANTILEVER	STANDARD PLAN #	UPRIGHT HEIGHT	UPRIGHT DIAMETER
ARM LENGTH	ARM DIAMETER	ARM CENTER TO CENTER	☐ GALVANIZED ☐ PAINTED
ARM CONNECTION GAPS	CANTILEVER LEANS (Towards or away from traffic, etc.)		ANCHOR BOLT PLAN
ANCHOR BOLT ID #	ANCHOR BOLT DIAMETER	BOLT CIRCLE DIAMETER	BASE DIAMETER

Figure 19.20 Example Bridge Mounted Sign Inspection Form

19.2.5 Common Mountings

Sign structures are typically mounted to bridges using one of two configurations:

1. Signs viewed from the under roadway can be mounted to the fascia girder and the side of the concrete deck or parapet in jurisdictions that permit this configuration (examples are provided in Figure 19.21). Concrete anchorages can be achieved through pre-installed anchors, grouted anchors or through-bolting; however the use of through-bolting should consider the potential for vehicular or plow blade damage from the over roadway. Steel anchorage should be accomplished with high-strength bolted connections to the fascia girder web or stiffener plate of the fascia girder. Welded connections are highly discouraged due to fatigue concerns. Where encountered, welded connections should be inspected regularly for cracking.
2. Signs viewed from the over roadway are typically mounted to the top of the concrete deck / parapet or cantilevered off the fascia girders (examples provided in Figure 19.22 (a) concrete bridge deck extension, (b) steel frame between 2 bridge fascia beams, and (c) steel bridge fascia beam).

Anchor rods are used to attach the structure to concrete bridge elements and high strength bolts are used to connect the structure to steel bridge elements.

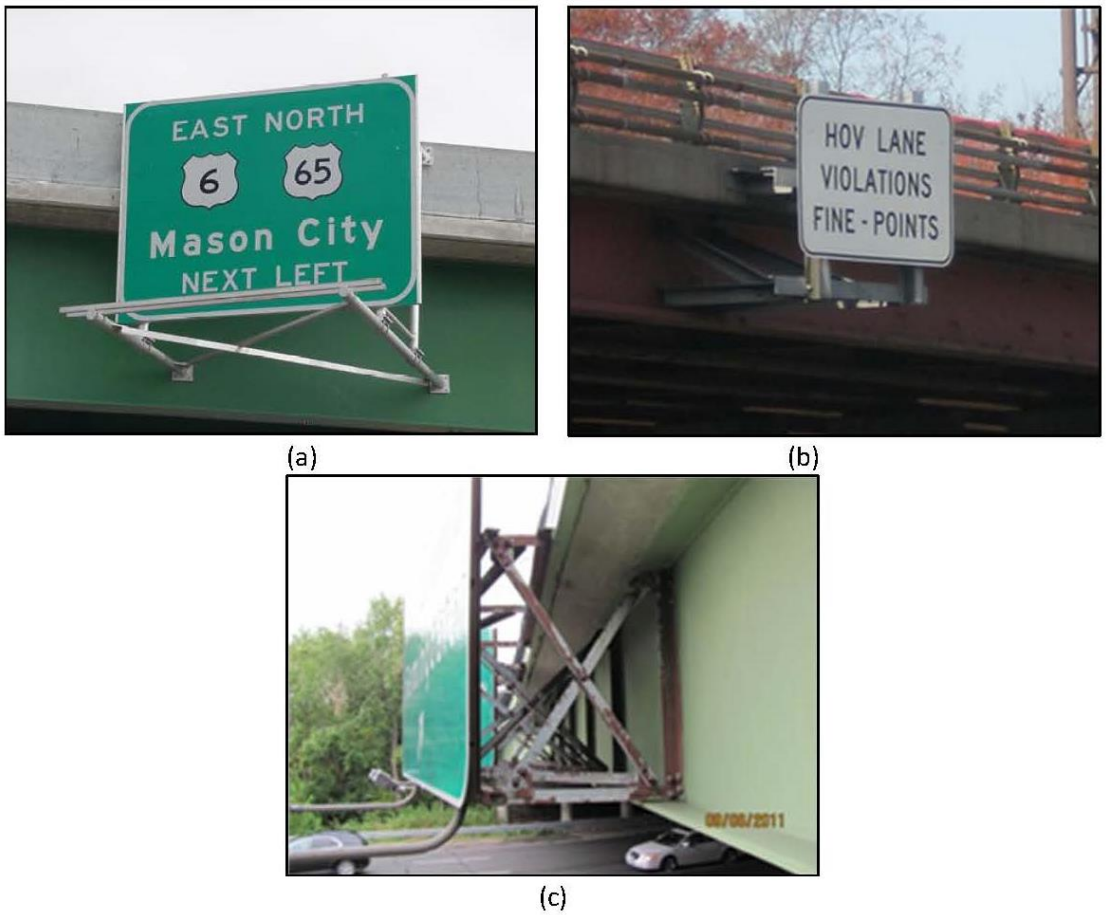


Figure 19.21 Examples of Sign Mountings for Viewing from Under Roadway

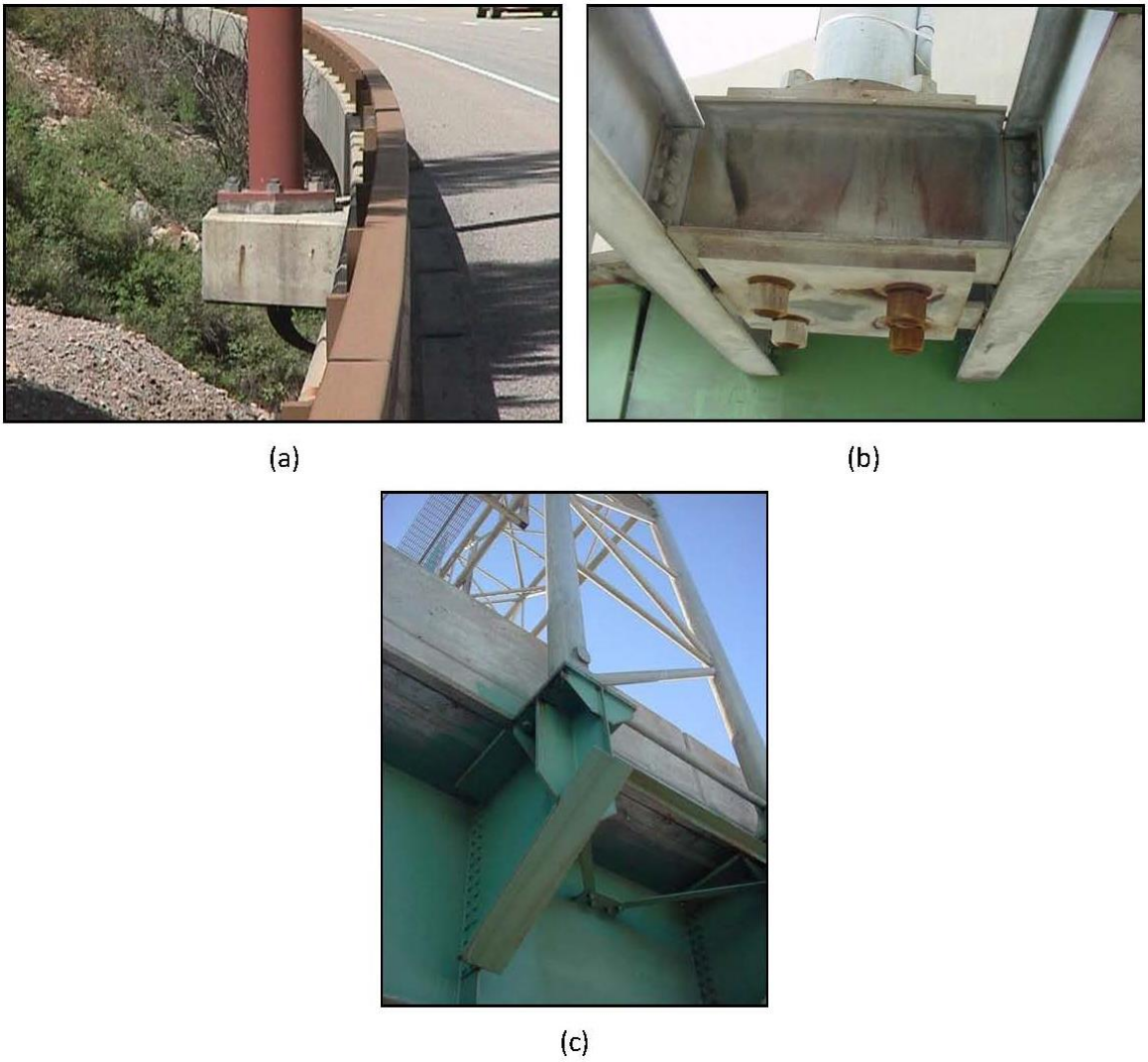


Figure 19.22 Signs Mounted to Various Bridge Components

Bridge mounted structures are subject to premature failure due to two primary mechanisms, as described below:

1. Repeated deflection of the supporting bridge elements under truck loading leads to additional fatigue cycles on the sign structure elements. This condition is more critical when the structure is mounted near mid-span locations.
2. Structures supporting signage viewed from the over roadway can be elevated a great distance above the surrounding ground level, subjecting them to increased wind velocity and pressure. Wind pressure is the primary load mechanism for sign support structures.

Bridge mounted sign structures may be located near electric facilities or be used as a conduit for electric circuits serving sign lighting or other attached devices. There have been situations where sign structures have been used as a conduit of convenience for electric crossings. This practice is not recommended as it presents an electrocution hazard if there is a high load impact. Proper clearances from sign structures to existing utility facilities should be checked and maintained as per established OSHA and National Electric Code guidelines to prevent potentially dangerous electrical shock or explosions. Prior to any work being performed, structures should be checked for stray voltage and communication with the associated utility

company should be made to obtain the necessary knowledge of voltages and work clearances to avoid potential hazards and code violations.

19.2.6 Bolts and Fasteners

High Strength Bolts, Anchor Rods, Stainless Steel or Aluminum Fasteners, U-Bolts, and Bolted Clips are some of the types of connection hardware used on sign structures. Structural connections generally utilize high strength bolts or anchor rods, while secondary connections are made with U-Bolts, Clips, Stainless Steel or Aluminum Fasteners. For an example of connection equipment, please see Figure 19.23. Installation tension requirements and corrosion potential should be considered when selecting connection hardware.

Figure 19.23 Example of Connection Equipment

Anchor Rods are generally set into the concrete slab or parapet, and are usually installed during the initial concrete pour. Leveling nuts are placed under the base plate and a top nut is installed with sufficient proof load to elongate the rod and create a permanent clamping force on the plate. Washers should be installed under both nuts and the “turn-of-the-nut” or other prequalified proof-load methods should be used to tighten double-nutted anchor bolt connections.

19.2.7 Tightness of Bolts

Sign structures on bridges are subject to constant vibrations due to fluctuating wind loads and bridge movements, sometimes leading to the loosening of connection hardware. In other cases, improper installation procedures can be the cause of loose hardware. Regardless of the cause, loose and missing hardware is of significant concern, as it can lead to a falling debris hazard, the overstressing of remaining connection elements (Figure 19.24) and reduced fatigue life for the connection. Sign structure failures are commonly attributed to loose anchor rod nuts leading to bolt fracture. When improperly installed, galvanized, high-strength bolts are difficult to tighten due to the loss of factory applied lubricants. Retightening efforts have been known to fracture anchor bolts. For this reason, it is recommended that loose nuts be replaced using the following repair procedure:

Suggested Procedure

Loose Anchor Bolt Nut Replacement

1. Provide temporary support in order to remove loading from the support.
2. Insert finger shims under any loose leveling nuts
3. Remove loose nuts (hydraulic nut splitter may be required)
4. Clean the exposed bolt threads with a wire brush and apply bees wax lubricant
5. Tighten the replacement nut using an approved proof load method (turn-of-the-nut, direct tension indicating washers, etc.)

Figure 19.24 Loose Anchor Bolts Revealed by Gap Under the Base Plate

Loose or missing hardware should be replaced and tightened with the proper lubrication and an approved tension verified proof-load tightening method (e.g., turn-of-the-nut method), unless it can be determined that sufficient redundancy exists in the connection. When possible, it is recommended to temporarily support the connected elements, remove the loose nut, clean and lubricate the bolt threads and install a new, lubricated nut. Proper nut tightening procedures will vary depending on the bolt material/diameter and the equipment used. Care should be exercised when tightening non-lubricated nuts to prevent permanent damage to the bolt due to thread and nut face friction.

If nuts are not fully engaged, they may not be able to transmit the intended load. An example of this situation is shown in Figure 19.25 (left). An example of a situation using supplemental anchor bolts is shown in Figure 19.25 (right).

Some or all of the following repair options may be available depending on the connection configuration.

Suggested Procedure

Partially Engaged Anchor Bolt Nuts – Repair Options

1. Evaluate not repairing if more than 75% of the threads are engaged.
2. Lower the base plate if anchor rod thread is available under the leveling nuts.
    a. Lower both sides of a span structure.
    b. Verify that minimum roadway clearances will remain available.
3. Remove the top of the parapet concrete and install a threaded coupler to extend the anchor.
4. Enlarge the base plate hole and install a specially machined shoulder nut.
5. Add additional fasteners (see Figure 19.25 right).
6. For bolts not embedded in concrete, such as a bridge mounted sign attached to a girder web, a simple solution might be replacing the bolt with a longer shank.

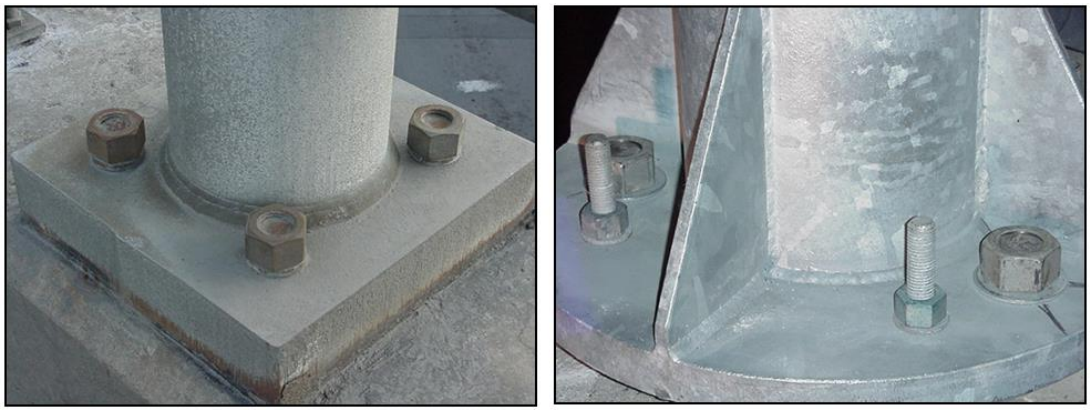


Figure 19.25 Examples of Nut Tightening Procedures

19.2.8 Epoxy Anchors

When to Call the Engineer

Call the Engineer if there is any indication of movement or damage to anchors or to surrounding substrate of anchors in continuous tension.

Fascia mounted sign structures are commonly anchored to the side of the concrete bridge deck or parapet rail. Anchor bolts set with epoxy (or other adhesives) into existing concrete can be considered to be a good, strong, connection if properly installed. When selecting anchorage options, it is important to ensure that the selected system is intended for the horizontally-inclined orientation under consideration. Installation of adhesive anchors subjected to continuous tension loading or used for overhead installations (see Figure 19.26) requires

specialized training. See the FHWA Technical Advisory *Use and Inspection of Adhesive Anchors in Federal-Aid Projects* at https://www.fhwa.dot.gov/bridge/t514030.cfm.

Recent failures of adhesive anchors in the transportation industry have focused attention on the training and certification of workers installing these systems. Certification programs are available through the Concrete Reinforcing Steel Institute. More information regarding certification programs can be found at the following web link: http://www.concrete.org/Certification/CertificationPrograms.aspx.

Figure 19.26 Example of Adhesive Anchors Subjected to Continuous Tension Loading

Upon identification of an anchorage failure (see example in Figure 19.27), a structure removal evaluation should be performed. If it is determined that the structure is safe to remain in service, repairs should be designed by a qualified professional engineer. Patching of ruptured concrete around an anchor is not an effective repair technique.

Figure 19.27 Epoxy Anchorage Failure

19.2.9 Dissimilar Metals

Utilization of various types of metal materials and connection hardware can lead to galvanic or bimetallic corrosion. This is a fast-acting, localized corrosion reaction which attacks the junction

or region where different construction metals are attached to one another. For this corrosion to occur, three conditions need to be present:

1. Metals are far apart on the galvanic scale (such as aluminum & steel)
2. Metals must be in contact with each other
3. The metal to metal junction is bridged by an electrolyte (such as water)

To protect against galvanic (bimetal) corrosion:

1. Don’t mix metals if at all possible
2. Prevent direct contact by using non-conductive spacers, gaskets, sleeves, etc.
3. Prevent moisture contact at the junction by installing a weather guard, using a sealant or protective coating, or providing proper drainage to guide water away from the junction.

19.2.10 Bridge Lighting

There are various types of lighting that may be present on and around a bridge structure. Lights may be mounted on poles to illuminate the over roadway, on concrete pier elements or structural steel elements to illuminate the under feature, as aids to waterway navigation, or as architectural adornment to illuminate the bridge itself. Some examples of these lighting situations are presented in Figure 19.28, including (a) pole-mounted over roadway, (b) mounted on pier or steel elements for under the roadway, and (c) architectural adornment.

(a) (b)

(c)

Figure 19.28 Examples of Various Types of Lighting

In addition to the common maintenance activities that bridge mounted lighting shares with bridge mounted signs (anchorages, connection bolts, etc.) agencies should plan for the additional requirements associated with electrified elements. These include repairs to damaged conduits, circuits, and lenses and the replacement of bulbs and other expired electrical components.

Bridge environments are collectors of road debris, animal nesting/droppings and road salts. Electric access locations should be sealed or otherwise isolated from these contaminants as well as from bridge washing activities to prevent deterioration and malfunction.

It is not uncommon for light poles to be fabricated from aluminum. Aluminum structures are susceptible to fatigue failure, which can be amplified when mounted to bridges (Figure 19.29 left). Aluminum welds should be visually inspected on a regular basis. Areas of concern include the pole-to-base connection and the hand-hole frame connection to the post (Figure 19.29 right). Visual inspection may be supplemented with non-destructive testing methods such as dye-penetrant or eddy-current. Magnetic Particle methods are not effective on aluminum structures.

Figure 19.29 Examples of Potential Areas for Fatigue Cracking in Aluminum Light Poles

19.2.11 Impact Damage

Collision or impact damage to a bridge lighting or sign system element needs to be inspected and evaluated immediately to determine if there is a hazard to the public (example shown in Figure 19.30). Common impact areas are the bottoms of sign panels and the posts. Damage may not be isolated to the immediate impact location. Care should be taken to inspect all load path components between the impact location and the bridge anchorage.

Common post-impact repairs include replacing broken hardware and replacing bent, cracked or torn structural elements. In many cases, a crane may need to be mobilized to remove damaged sign panels or the entire structure from service.

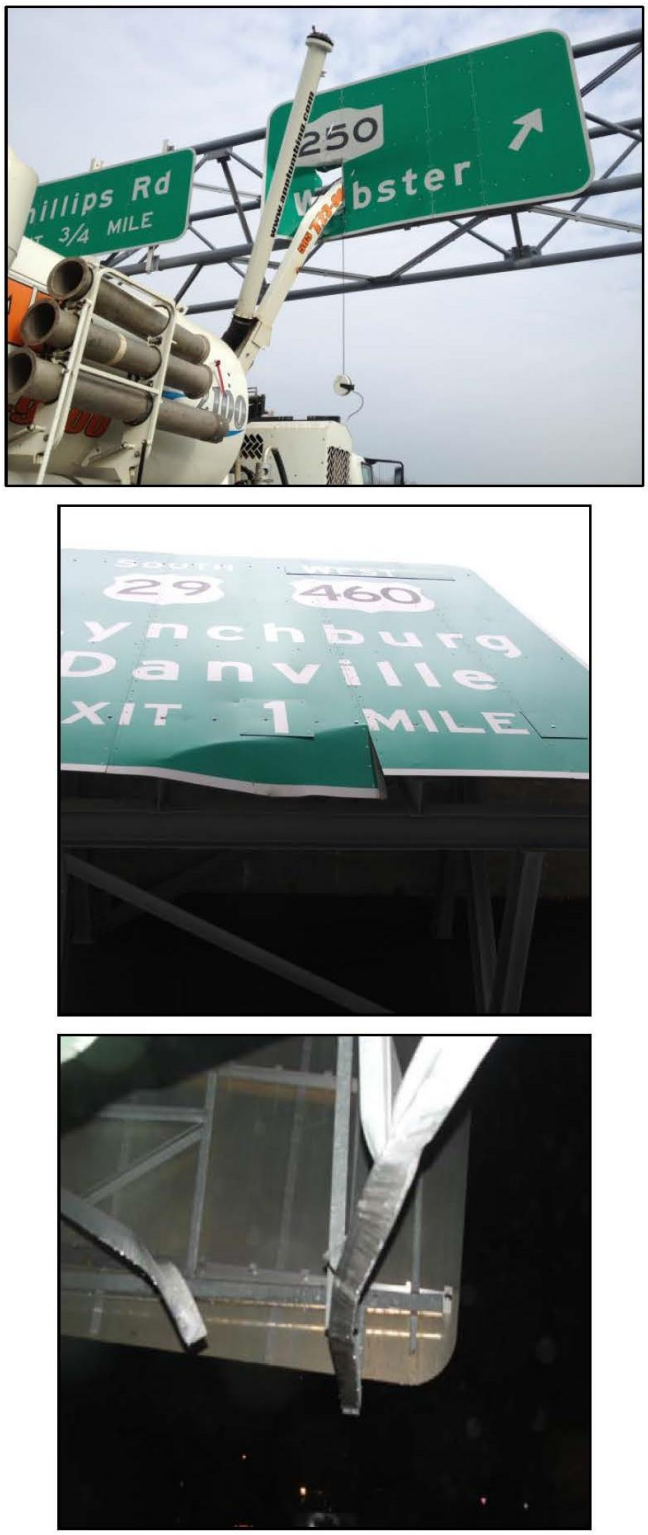


Figure 19.30 Examples of Impact Damage to Sign System Elements

19.2.12 Fiber Reinforced Polymer (FRP) Repairs

FRP products are an effective way to repair steel members damaged by corrosion or aluminum welds damaged by fatigue. Oxygen is the fuel for corrosion action and encasement with an FRP product can both strengthen and isolate the element from future corrosion.

FRP repairs have been effectively used to repair cracked secondary sign truss connections with minimal disruption to traffic, an example of which is shown in Figure 19.31. Originally intended as a temporary solution, non-destructive testing has proven the method's long term performance. An NDT example of an FRP crack repair is shown in Figure 19.32.

Figure 19.31 FRP Repair of Cracked Secondary Truss Connection

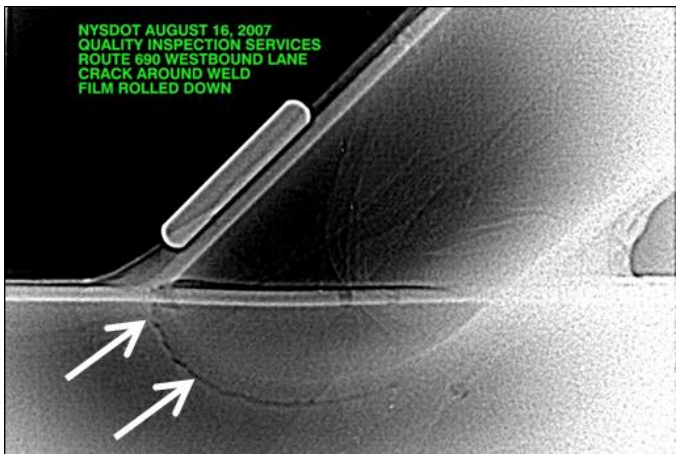


Figure 19.32 NDT of FRP Repair. Crack at Weld Indicated by Arrows.

When to Call the Engineer

If FRP repair is being installed, the Engineer should be consulted for the design of the repair.

FRP structural repairs should be designed by a qualified professional engineer to provide a strength equal to or greater than the member being repaired. The repair should be performed in accordance with the FRP manufacturer's recommendations. As FRP repairs age, repairs to the wrap may be required. Typical repair procedures for FRP wraps are detailed below.

Suggested Procedure

Repair of Surface Cracks or Damage in Existing FRP Wrap

1. Obtain a repair procedure from a qualified engineer.
2. Clean the surface with a solvent approved by the FRP manufacturer.
3. Apply a resin approved by the FRP manufacturer.
4. Apply one layer of compatible FRP over the existing FRP using manufacturer's recommended installation methods.
5. If the resin does not contain an ultraviolet light (UV) inhibitor, apply an opaque, weather resistant, UV protective paint compatible with the FRP materials.

Suggested Procedure

FRP Wrap – Repair for Delamination Between Wrap and Underlying Metal

1. Obain a repair procedure from a qualified engineer.
2. Remove the existing FRP wrap from the structure, being careful not to damage the base metal.
3. Prepare the surface in accordance with the manufacturer's installation procedures.
4. Apply new FRP material repair materials in accordance with the manufacturer's recommendations.

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Chapter 20 - Concrete Deck Evaluation Procedures and Decision Aid Matrices

20.1 Reinforced Concrete Bridge Deck Evaluation Introduction

The evaluation of a reinforced concrete bridge deck involves using the information available from visual inspection, non-destructive testing, and material tests to determine appropriate repair or preservation activities. Deck evaluations can be complex due to the number of variables that must be considered to determine the appropriate repair or treatment method. The following chapter describes the tools and techniques used to gather deck information, a description of the key deck decision variables, and a reinforced concrete deck evaluation methodology.

For additional evaluation and treatment methods, please see:
http://maintenance.transportation.org/documents/final_nchrp%20guidelines%2020-07%20task234_wje%20_comments%205-19-09.pdf

20.2 Deck Investigation Techniques

20.2.1 Introduction

Reinforced concrete bridge deck investigations should be carried out in a progressive manner as conditions or field findings warrant. The deck investigation progression typically begins with a visual assessment of the condition of the deck. The visual condition assessment consists of looking for cracks and spalls or other visible distress on the top and bottom surface of the deck or slab. In some cases, a deck delamination survey may be conducted during a routine NBI bridge inspection. The delamination survey typically consists of chain or hammer sounding or NDE testing to determine the delaminated concrete area and may cover portions or the entire surface of the deck. If the visual assessment and delamination survey reveal no issues then further investigation is not typically warranted. If on the other hand, the visual condition assessment and delamination survey identifies conditions that warrant further evaluation, the investigation can progress to include more detailed field tests, and/or material sampling to determine the appropriate repair or preservation treatment.

20.2.2 Preparing for a Reinforced Concrete Deck Investigation

A reinforced concrete bridge deck investigation should begin with a review of as-built plans and the history of prior inspections and testing that may have been done. The agency's records of prior NBI bridge inspection reports and any deck testing reports for the bridge should be consulted. The as-built plans and inspection history should be reviewed to determine as many of the items shown in Table 20.1 as possible. An engineer supporting the bridge maintenance activities should be consulted for information related to past inspection information and past testing results and for a recommendation of the appropriate deck investigation strategy.

Table 20.1 Deck Investigation Preparation

Deck Attributes	Key Information
Type of Reinforcing Steel	What is the reinforcing material (black steel, coated steel, galvanized steel, stainless steel, fiber reinforced polymer)? The reinforcing type will impact corrosion rates.
Material Specifications	What is the 28 day compressive strength of concrete (f'c) and the yield strength of the reinforcing (fy)?
Reinforcing layout	What is the size and spacing of the deck reinforcing? Some deck cracking will follow reinforcing lines.
Design Concrete Cover	How thick is the concrete protection over the top of the reinforcing? Concrete cover can impact corrosion rates.
Deck Structural Thickness	What is the overall deck section thickness? The overall thickness can influences deflection and cracking.
Girder/Deck Connection	Is the deck composite (i.e., rigidly attached to the girders with studs) or non-composite (i.e., not rigidly attached)? Deck performance may be impacted by the connection to the superstructure.
Girder Spacing	Identify the distance that the deck must span between girders. The deck must transfer wheel loads laterally to the girders. The girder spacing can influence deck deflections and cracking.
Prestressed or Post-Tensioned Girders	Are the girders prestressed or post-tensioned? Stressed girders may creep and shorten and could cause deck cracking.
Overlays, Membranes, Cathodic Protection Systems	Any records related to wearing surface materials, membranes or cathodic protection systems and placement dates should be noted.
Widening History	Are all the sections of the deck the same? Widenings introduce the potential for varying condition and performance limits over the deck area.
Surface Drainage Information	Where does all the water on the deck go? Is the deck crowned or on a super-elevation? Where are the longitudinal drains? Does the deck experience snow, salting, freezing temperatures?
Deck Inspection History	Review the inspection history to identify when deck distress may have been first noticed and the rate of progression of the deterioration.
Previous Material Tests	Review any previous deck material test. The previous tests can provide baseline results and possibly help explain the cause of field observed conditions.
Maintenance and Repair Records	Review the maintenance history for the bridge previous actions taken, products used and performance of past products.

20.2.3 Field Tools

The bridge inspection records and as-built plan review will usually provide clues to help determine what tools will be necessary for an in depth deck investigation. The following section describes the tools typically used for the inspection and testing of reinforced concrete bridge decks:

- Geology Pick or Hammer – The geology pick is commonly used to tap localized areas of a bridge deck to listen for potential delaminations.
- Rag Tape or Rolling Distance Measurement Device – A distance measuring device is necessary to help lay out a grid on the deck so that identified defects and test measurements can be located within the deck area.
- Crack Comparator – A crack comparator is a clear plastic credit card sized sheet with crack sizes shown along the edges. The crack comparator is placed on the concrete crack and moved up or down the crack scale until the exact crack with is determined. An example of a crack comparator is presented in Figure 20.1.

Figure 20.1 Crack Comparator

- Deck Chain – Delamination surveys done on larger areas of bridge decks are typically done with a deck chain. The deck chain may be a simple single chain or a T-bar with a number of short chain stands that hang off the bottom of the bar. The deck chain is dragged across the bridge deck in a systematic manner while the deck investigator listens for frequency changes in the resulting sound. Delaminated concrete can be readily identified by the hollow sound that emanates from the concrete when the chain passes over the delamination. An example of a T-bar deck chain is presented in Figure 20.2. (Deck chaining was previously presented in Section 5.4.3 Non-Destructive Evaluation (NDE) Techniques.) Deck chaining however is not effective if there is an asphalt riding surface.

Figure 20.2 T-bar Deck Chain

- Ground Penetrating Radar (GPR) – GPR equipment can be used as an alternative to deck chaining or impact echo methods to determine the areas of delaminated concrete. The GPR method involves sending electromagnetic waves into the deck surface from the GPR antenna and measuring the return strength. The internal makeup of the deck being investigated can then be mapped including rebar locations, depths, concrete cracking and voids. An example of a ground penetrating radar schematic is shown in Figure 20.3, and a van-mounted ground penetrating radar system is shown in Figure 20.4. GPR was also discussed in Sections 5.4.3.7 Advanced NDT Techniques and 16.2.1 Inspection of Masonry Bridges.

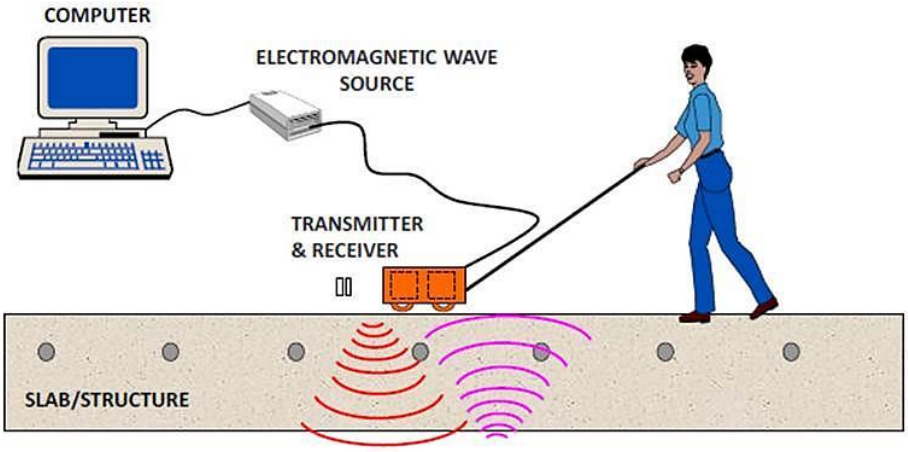

Figure 20.3 Ground Penetrating Radar Schematic

Figure 20.4 Van Mounted GPR

- Impact Echo Testing – An impact echo device is used to identify and locate the limits of delaminations below the concrete surface. The impact echo device mechanically impacts the deck at grid points and records response sound waves (echoes) that bounce back from internal flaws before reaching the bottom edge of the deck. Delaminations internal to the deck will cause the sound to bounce back faster than expected allowing the computer to determine the depth of the flaw. Impact echo testing has been proven to be very effective at locating delaminations in concrete. An example of impact echo testing equipment is shown in Figure 20.5. Impact echo testing was also discussed in Sections 5.4.3.7 Advanced NDT Techniques and 16.2.1 Inspection of Masonry Bridges.

Figure 20.5 Impact Echo Testing Equipment

- Coring Equipment – Drilling and coring equipment may be necessary to collect concrete samples for laboratory testing or to expose reinforcing steel for field corrosion testing. An example of concrete deck coring equipment is shown in Figure 20.6. Coring was also introduced in Section 5.4.4 Destructive Testing.

Figure 20.6 Concrete Deck Coring Equipment

- Pachometer – A handheld pachometer (i.e., cover meter) is typically used to locate reinforcing steel beneath the concrete deck surface. The pachometer can also determine the depth of concrete cover over the steel without disturbing the concrete. An example of a pachometer device is shown in Figure 20.7. The pachometer was previously introduced in Section 5.4.3.4 Corrosion Survey in Chapter 5 – Concrete Basics.

Figure 20.7 Pachometer Device

- Potentiometer – A potentiometer will be necessary if the investigation includes half-cell corrosion potential testing. The potentiometer may be a handheld digital device or tube device with a wire and probe on one end and a rebar attachment device on the other end. The current between the concrete and steel is measured. An example of a potentiometer is shown in Figure 20.8.

Figure 20.8 Potentiometer

20.2.4 Traffic Management

Detailed deck investigations typically require lane closures to properly assess the condition of the deck. Most agencies have standard lane closure plans that can be used to safely shut down one or more lanes on the bridge for the deck investigation. The Federal Highway Administration also produces the Manual for Uniform Traffic Control Devices (MUTCD) that provides traffic control plans for commonly occurring closure situations.

20.2.5 Inspection Grid Layout

A deck investigation requires detailed note taking of field measurements and conditions. The best way to get accurate field investigation notes is to lay out a grid pattern on the deck to demark smaller sections of the deck that can be individually assessed and reported. The size of the grid can vary depending on the nature of the distress being investigated and the overall size of the deck. Common grid layouts range from 2 by 2 feet to 5 by 5 foot grids. The accuracy of some field measurements, such as half-cell testing, can have significant loss of accuracy as the grid size increases from 2 feet to 5 feet.

The grid can be established in the field by marking fixed intervals on the deck with spray paint or chalk using a rag tape measure or rolling distance measuring device (see Figure 20.9). Once the grid has been laid out within the closure limits, the investigation can begin. Notes are taken grid by grid to quantify field measurements and distress within the defined localized area of each grid. The investigation progresses until all grids of the deck have been investigated. Each grid assessment is like a single piece in a puzzle, when all the grid results are documented a picture of the extent and nature of distress emerges.

Figure 20.9 Field Grid Layout

20.2.6 Visual Deck Inspection

Visual inspection of reinforced concrete bridge decks provides the inspector with information that can be used to determine if more in-depth investigation or testing is warranted. Visual indicators of bridge deck deterioration or potential for deterioration include: cracking, spalling, abrasion, scaling, efflorescence, rust staining, and freeze thaw damage.

What To Look For

Indicators of bridge deck deterioration, such as cracking, spalling, abrasion, scaling, efflorescence, rust staining, and freeze thaw damage.

20.2.6.1 Deck Cracks

The severity of deck cracking is classified based on the frequency, width and length of the cracks. Deck cracks can be classified into several basic types generally based on the orientation of the cracking or the crack patterns produced. Typical crack types and causes are shown in Table 20.2.

Table 20.2 Typical Crack Types and Causes

Crack Type	Cause	Orientation
Shrinkage cracks (Plastic and Drying Shrinkage)	Concrete drying too quickly	Well-defined widely spaced pattern with shallow and wide cracks
Stress Cracks	Stresses in the concrete exceeding tensile capacity	Pronounced on stress lines, typically transverse or diagonal
Settlement Cracks	Settlement of hardened concrete	They can be of any orientation and width
Map Cracks	Reactions between the cement and aggregate (alkali-silica and alkali-carbonate reactions)	Map cracks are a closely spaced uniform pattern over large areas
Corrosion Induced Cracks	Expansion of corroding reinforcement	Oriented along reinforcement lines
Restricted Movement Induced Cracks	Temperature expansion and contraction of concrete that is restricted	At acute corners of skewed bridges

Regardless of cause, reinforced concrete bridge deck cracking creates potential pathways for water and salts to penetrate the concrete matrix and cause deterioration of the concrete or reinforcing steel. Minimizing reinforced concrete deck cracking through proper mix design, placement and curing methods will improve the service life of the deck. When cracks occur, sealing the cracks or applying other preservation treatments that minimize the ingress of water and salts will extend the life of the bridge deck.

20.2.6.2 Deck Spalls

Decks spalls can be caused by pressure exerted by the corrosion of reinforcement and the resulting expansion that causes cracking beneath the surface of a bridge deck in a plane parallel to the roadway surface. The cracked concrete is pounded by traffic ultimately causing it to break off completely as a spall. The spalled area left behind is characterized by sharp edges on the sides and bottom of the area. Spalls can also be caused by vehicular forces on exposed concrete edges such as deck joints or construction joints. Additionally, spalling may be caused by overloading of the concrete in compression (commonly from restricted thermal expansion). This results in the breaking off of the concrete cover to the depth of the outer layer of reinforcement.

Spalls are visible during a deck inspection and are typically reported using the area spalled out and the depth of the spall. Spalls often occur in areas where delaminations are occurring. The area around a spall should be hammer tapped or chained to determine if there is any delaminated area outside the limits of the spall. Delaminated areas surrounding a spall should be chipped out and patched when the spall is repaired. For more information regarding concrete repair, please refer to Section 11.1.3 Spall and Crack Repair and 11.5.3 Concrete Superstructure Patching.

20.2.6.3 Abrasion and Rutting

Abrasion of the top surface of concrete bridge decks can occur from the wearing action of studded tires, snow chains and snow plows. Abrasion is most often found in the wheel lines of the lanes only. The abrasive action slowly chips away the top layer of cement in the concrete exposing the aggregate below the surface. If abrasion is allowed to progress over time, ruts can develop in the wheel lines of the deck. Rutting is characterized by shallow depressions in the wheel lines caused by the concrete being worn away from abrasion. If left untreated, abrasion and rutting tend to attract water and salts into the depression, and expose the concrete matrix which may accelerate the ingress of chlorides.

20.2.6.4 Scaling

Scaling is the deterioration of the upper quarter to half inch of the concrete deck surface and is characterized by the peeling or flaking away of the surface concrete. Scaling results from repeated freezing and thawing of concrete at high moisture levels. Freezing water held in the concrete expands and causes deterioration of the concrete. The effects of scaling are accentuated by the presence of deicers and salts. Scaling decks will often have the course aggregate exposed and some of the aggregate may have been dislodged by the lack of sound concrete paste holding it in place.

20.2.6.5 Efflorescence

Efflorescence is a white crystal residue that results when salt laden water reaches the surface of the concrete, evaporates, and salt deposit is left behind. There are different types of salts, and efflorescence on bridges is most often caused by salts that were in the original concrete ingredients. Efflorescence is common on the underside of reinforced concrete bridge decks at cracks or other areas of high permeability. Although efflorescence is harmless, it is an indicator that water is moving through the deck at that location, and the water is likely to contribute to reinforcing steel corrosion and possibly freeze/thaw damage. Advanced efflorescence may result in crystallized deposit building up over time to form stalactites that protrude down from the underside of the bridge deck. A photographic example of efflorescence is shown in Figure 20.10.

Figure 20.10 Typical Soffit Crack with Efflorescence

20.2.6.6 Rust Staining

Rust staining visible on the top surface, or soffit, of the deck is a clear indication that corrosion is actively occurring within the reinforced concrete deck. Rust staining on the soffit is also an indication of areas where water is passing through the deck from above. The location of all rust staining should be noted during the deck investigation to support other evidence of reinforcing steel corrosion at that location.

20.2.6.7 Follow the Water

Water is a conveyance method for chlorides and can cause deterioration in the concrete on its own. When conducting a deck investigation, it is important to understand the flow of surface water. By following the water path, the investigator can quickly locate high deterioration risk areas on the deck. When in the field, take note of the cross slope of the deck. Does the water flow to one side or both? Note how the water flows along the length of the bridge. Are there deck drains at low points along the deck or does the water flow off the bridge? If there are construction joints that may be more permeable than the surrounding deck concrete, focus attention there particularly if the construction joint is at a low point in the deck. The investigator should also look at the profile of the deck and rails to identify any potential low points that resulted from poor grade setting during construction. The low points will tend to have more moisture and chlorides than other areas of the deck, increasing chances of deterioration.

20.2.6.8 Freeze Thaw Damage

Freezing causes a nine percent expansion in the volume of water held in concrete cracks and pores. The freezing expansion of the water can cause pressure to build up in the cracks and pores causing localized fractures. Freeze thaw damage is usually isolated near the top surface or edge of the deck.

20.2.6.9 Evaluating Decks with Wearing Surfaces

When the bridge deck top surface is covered with an overlay, visual inspection cannot quantify cracking or spalls unless they reflect through the wearing surface material. In these situations, other indicators of deck deterioration must be used to assess the condition under the wearing surface. Cracking in the underlying deck is not of significant concern if the wearing surface is protecting the deck and does not have cracks or spalls, and if the wearing surface is low permeability or impervious. If the wearing surface is cracked, spalled, or delaminated, these conditions can be addressed using many of the same sealing and patching methods used for decks without wearing surfaces. Soffit conditions such as spalling, cracking and efflorescence or rust staining can provide clues to the condition of the deck under the wearing surface. If the underlying deck is suspected to have significant deterioration under the wearing surface, sections of the wearing surface can be removed for visual inspection. Alternately, core testing can be conducted to ascertain the deck condition.

20.2.6.10 Evaluating Decks with Stay-in-Place Forms

Stay-in-place forms are used in many areas to speed up forming and construction of bridge decks. Stay-in-place forms eliminate the possibility of visually inspecting the underside of the deck for cracking, efflorescence, rusting and spalling. If the top surface is visible for inspection, the deck investigation should use only to the top surface visual findings. Sometimes, the top

surface is covered with a wearing surface and the soffit is covered with stay-in-place forms. In this case, the deck investigation must rely on non-destructive testing methods and coring to determine the condition of the deck sandwiched between wearing surface and stay-in-place forms.

20.2.7Material Testing

20.2.7.1 Determining Chloride Content

Reinforced concrete bridge decks exposed to deicing salts and seawater are vulnerable to the ingress of chloride ions. High chloride content in concrete encourages corrosion of steel reinforcement by destroying the passive protective layer on the reinforcement. Loss of the passive protective layer leads to corrosion in the reinforcing bar. The most damaging chlorides can be found in the concrete pore water and travel through cracks and voids in the concrete matrix. The application of deicing salts causes ice and snow to melt creating a highly concentrated chloride solution. The chlorides in the pore water are the principal cause of breakdown in the passive oxide film protecting the reinforcement from corrosion.

20.2.7.2 Determining pH Levels

The pH levels in concrete can be lowered to levels that cause the reinforcing steel to corrode through carbonation of the concrete. In the presence of moisture, carbon dioxide from the air reacts with materials in the concrete to form calcium carbonate and carbonic acid. Both of these products can lower the alkalinity of normal concrete, so that the pH is below the level at which the protective oxide film on the reinforcing steel is stable. Without the protective oxide film, the reinforcing steel begins to corrode. The degree of carbonation can be determined by measuring the amount of calcium carbonate through a petrographic analysis (petrographic analysis will be discussed in Section 20.2.7.4). The depth of carbonation is measured with a phenolphthalein color test, which determines the pH of concrete. In poor-quality, carbonated concrete (pH of 9.0 to 9.5), the phenolphthalein solution remains colorless. In high quality, noncarbonated concrete (pH of 12.5 or greater), the solution turns pink. An example of a petrographic cross section used to measure carbonation depth is shown in Figure 20.11.

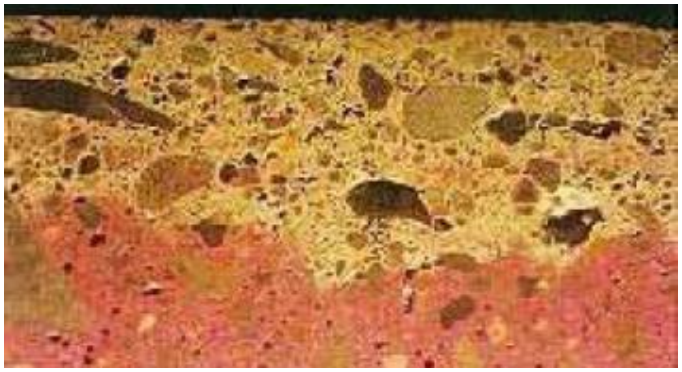

Figure 20.11 Phenolphthalein Reveals Noncarbonated Areas

20.2.7.3 Determining Concrete Compression Strength

Bridge deck concrete has specified minimum compression strength requirements that typically range from 2,500 psi to 5,000 psi after a 28 day cure. The compression strength of any in-situ concrete deck can be evaluated by extracting cores and testing the material to failure in the lab.

The lab will report the compressive stress that caused the core sample to fail. The results from the lab can be compared to the design compressive strength to determine if the capacity of the deck has been reduced. Compressive strengths of concrete can diminish over time due to alkali silica reactivity (which will be discussed in Section 20.3.4), freeze thaw damage, and excessive cracking.

20.2.7.4 Petrographic Analysis

A petrographic analysis looks at the microscopic structure of the concrete using a petrographic microscope. Very thin slices of concrete from a concrete core sample are examined to determine characteristics of cracking, air entrainment, reactive aggregates, effect of fire exposure, carbonation, and many other properties of the hardened concrete. Petrographic analysis is very thorough but is significantly more expensive than other test methods used for concrete deck investigations. The high cost of the petrographic testing typically limits the use of this test to decks first evaluated using other investigation methods and found to have problems. An example of a petrographic analysis of a shrinkage crack in concrete is shown below in Figure 20.12.

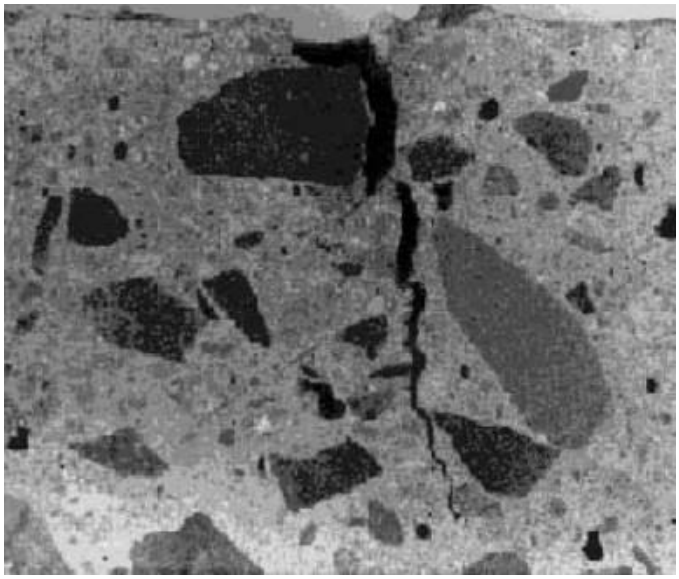

Figure 20.12 Petrographic Analysis of a Shrinkage Crack in Concrete

20.2.7.5 ASR Testing Methods

There are two common methods used to determine if ASR is present in concrete: uranyl-acetate testing and petrographic analysis. Both tests require concrete core samples to be taken for analysis in the lab. The uranyl-acetate test involves treating a core sample section surface with uranyl-acetate. ASR present in or around the aggregate will fluoresce under ultraviolet lights. Petrographic analysis is the most reliable test method for determining ASR.

20.3 Evaluating Test Results

20.3.1 Compressive Strength

Compressive strength results collected during a deck investigation should be compared to the design concrete strength. If the tested concrete strength is below the design strength specified on the plans, an analysis of the member capacity should be undertaken using the tested

concrete strength. If the structure is found to have reduced capacity at the tested concrete strength, then petrographic analysis would be warranted to determine the cause of the loss of strength. If the concrete compressive strength is found to be acceptable through analysis, but is below the specific design strength, the deck should be protected from further deterioration of the concrete.

20.3.2 Chloride Content

Chloride content is an indicator of the potential for corrosion. The presence of chlorides and water provide the necessary ingredients for corrosion in a reinforced concrete bridge deck. The generally accepted chloride levels that can cause corrosion in reinforcing bars are shown in Table 20.3.

Table 20.3 Impact of Chloride on Potential for Corrosion

Potential for Corrosion	Chlorides at Reinforcing (Black Bars) Pounds per Cubic Yard	Chlorides at Reinforcing (Coated Bars) Pounds per Cubic Yard
Low (< 10%)	< 2.0	< 10
Moderate (~ 50%)	2.0 - 2.5	10 - 15
High (> 90%)	> 2.5	> 15

Note: 1 pound/cubic yard = 255 parts per million

20.3.3 Half-Cell Potential

Half-cell testing, when performed on bridges, measures the electrical potential between the reinforcing and the concrete. The half-cell device consists of a reference electrode, a sensitive volt meter, and a data logger. The negative end of the volt meter is connected directly to the reinforcement by drilling a hole in the deck. The electrode is placed on top of the concrete deck at grid points and the corresponding voltage is measured. It is important when doing half-cell testing to use a grid that does not exceed 2 feet by 2 feet. The more negative the voltage reader the greater the potential for corrosion. The half-cell results typically consist of a corrosion potential map similar to the one shown in Figure 20.13. It should be noted that the second y-axis in the figure is presented in millivolts (not volts).

The corrosion potential map can be used to identify areas that are more prone or less prone to corrosion and should have correspondence to any identified delaminations. For epoxy coated bars, the potential should be significantly less than uncoated bars because the epoxy coating will not conduct electricity. Half-cell testing results of epoxy coated steel decks should be closely evaluated for validity and repeatability given the relatively low corrosion potential. If the epoxy coating has been damaged during placement or through deterioration, the half-cell potential will become more negative (increase). If the half-cell potential levels reach levels defined in Table 20.4, corrosion can occur even though the bars are coated.

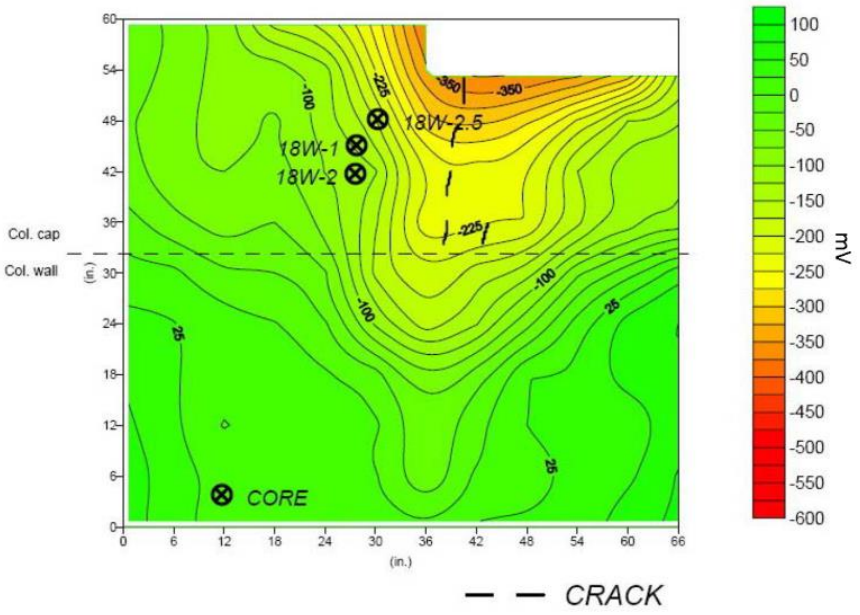

Figure 20.13 Half-Cell Potential Map

Table 20.4 Potential for Corrosion

Potential For Corrosion (Coated and Uncoated Bars)	Concrete Half-Cell Potential (Volts)
Low	More positive than -0.20
Moderate	Between -0.2 and -0.35
High	More negative than -0.35

20.3.4 Alkali Silica Reactivity (ASR)

Alkali Silica Reactivity (ASR) affects concrete structures in almost every area of the U.S. It occurs when silica in the aggregate and alkali in the cement react in the presence of water. This reaction creates a gel in the hardened concrete. Over time, this gel expands causing closely spaced map cracking and disintegration of the bond between the concrete ingredients. Most agencies have implemented material specifications that limit the potential on new bridges, however many older bridges may not have had similar specifications in place to control ASR.

There are two common methods used to determine if ASR is present in concrete: uranyl-acetate testing and petrographic analysis. Refer to Sections 20.2.7.4 and 20.2.7.5 for more information on these lab tests.

ASR test results will confirm the presence of ASR. If reactive aggregates are present, the test will typically classify the concrete as having the potential for ASR or having active ASR. The potential for ASR usually means that the aggregate has the potential to react, but the gels produced from reaction are not evident. Active ASR results indicate that reactive aggregates are present in the concrete and that these aggregates have already produced expansive reactive gel by-products. Positive test results for ASR would typically require protective treatment or replacement of the concrete in severe situations.

20.4 Deck Evaluation Decision Matrices

20.4.1 Deck Evaluation Methodology

There are two broad categories of reinforced concrete bridge deck deterioration: corrosion and material breakdown. A systematic process to evaluate the most common and relevant bridge deck variables for each deterioration category is presented in this section.

20.4.2 Determining Potential for Corrosion

Corrosion potential can be influenced by cracking, half-cell results, chloride content, pH levels in the concrete, and rebar cover depths. Although not an ideal situation, the potential for corrosion can be determined without knowing all the attribute information. Table 20.5 and Table 20.6 can be used to assess the risk of deterioration of reinforced concrete decks with uncoated and coated reinforcing. For decks with stainless steel or galvanized reinforcing steel, use Table 20.6.

It is possible that deck investigation results will have attributes that fall into multiple corrosion potential classifications. If this is the case, use the most aggressive corrosion classification.

Table 20.5 Uncoated Deck Reinforcing Corrosion Potential Classification

Test Result	Low Potential	Moderate Potential	High Potential
Chloride Levels (Pounds per Cubic Yard)	< 2	2.0 to 2.5	> 2.5
Half-cell Potential (Volts)	More positive than -0.2	-0.2 to -0.35	More negative than -0.35
Cover Concrete Depth (Inches)	> 1.9	1.0 to 1.9	< 1.0
Concrete pH	> 9.0	7.0 to 9.0	< 7.0

Table 20.6 Coated Deck Reinforcing Corrosion Potential Classification

Test Result	Low Potential	Moderate Potential	High Potential
Chloride Levels (Pounds per Cubic Yard)	< 10	10 to 15	> 15
Half-cell Potential (Volts)	More positive than -0.2	-0.2 to -0.35	More negative than -0.35
Cover Concrete Depth (Inches)	> 9.0	7.0 to 9.0	< 7.0

The corrosion potential classification results above can be used to select the appropriate action table (Table 20.7 through Table 20.9). The action tables should be used with the results of the

visual deck assessment of cracking and spalls and the delamination survey. The visual assessment information will intersect on a recommended action class.

20.4.3 Reinforced Concrete Deck Action Classes

Having assessed the corrosion potential of the deck being investigated, select the appropriate action class table for low, moderate, or high corrosion potentials determined in Section 20.4.2. Use investigation results for cracking, spalls and delaminations to identify the appropriate action class provided in Table 20.7 through Table 20.9.

The cracking attribute should reflect the predominate size and spacing over the entire deck or over the localized area of concern if the deck investigation or repair is limited to certain areas of the deck. In general, the potential for water and salts to enter the deck matrix is being evaluated with the crack information. Spalls and delaminations, on the other hand, are damage that has already occurred.

Table 20.7 Low Corrosion Potential Actions

	Percent Spalls and Delaminated Deck Area 0% < Distress <2%	**Percent Spalls and Delaminated Deck Area 2% < Distress < 5%**	**Percent Spalls and Delaminated Deck Area 5% < Distress < 10%**	**Percent Spalls and Delaminated Deck Area Distress > 10%**
Deck Cracking Width < 0.02 inch AND Spacing > 3 feet	Do Nothing OR Repair	Repair	Repair	Rehab OR Replace Deck
Deck Cracking Width < 0.02 inch AND 1 foot < Spacing < 3 feet	Do Nothing OR Repair	Repair	Repair	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch AND 1 foot < Spacing < 3 feet	Do Nothing OR Repair	Repair AND Seal Deck	Repair AND Overlay	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch AND Spacing < 1 foot	Repair AND Seal Deck	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck

Table 20.8 Moderate Corrosion Potential Actions

	Percent Spalls and Delaminated Deck Area 0% < Distress <2%	Percent Spalls and Delaminated Deck Area 2% < Distress < 5%	Percent Spalls and Delaminated Deck Area 5% < Distress < 10%	Percent Spalls and Delaminated Deck Area Distress > 10%
Deck Cracking Width < 0.02 inch AND Spacing > 3 feet	Repair	Repair	Repair AND Seal Deck	Rehab OR Replace Deck
Deck Cracking Width < 0.02 inch AND 1 foot < Spacing < 3 feet	Repair AND Seal Deck	Repair AND Seal Deck	Repair AND Seal Deck	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch AND 1 foot < Spacing < 3 feet	Repair AND Seal Deck	Repair AND Protect Deck	Repair AND Overlay	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch AND Spacing < 1 foot	Repair AND Seal Deck	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck

Table 20.9 High Corrosion Potential Actions

	Percent Spalls and Delaminated Deck Area 0% < Distress <2%	Percent Spalls and Delaminated Deck Area 2% < Distress < 5%	Percent Spalls and Delaminated Deck Area 5% < Distress < 10%	Percent Spalls and Delaminated Deck Area Distress > 10%
Deck Cracking Width < 0.02 inch AND Spacing > 3 feet	Repair AND Seal Deck	Repair AND Seal Deck	Repair AND Seal Deck	Rehab OR Replace Deck
Deck Cracking Width < 0.02 inch AND 1 foot < Spacing < 3 feet	Repair AND Seal Deck	Repair AND Seal Deck	Repair AND Overlay	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch AND 1 foot < Spacing < 3 feet	Repair AND Seal Deck	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch AND Spacing < 1 foot	Repair AND Overlay	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck

20.4.4 Selecting the Appropriate Deck Actions

Having determined the appropriate action class from Table 20.7 thru Table 20.9, use the following discussion to further refine the action class for determining the specific deck repair or treatment.

Do Nothing

- **Low Corrosion Potential** – The likelihood of reinforcement corrosion is minimal in these decks. For decks with minimal deck spall and delamination damage, it may be advisable to do nothing if the area of repairs is on the low end of the given range or if there is any uncertainty in the delamination determination. The cracking does not need to be treated until it increases in size or spacing is reduced. However, in areas where deicing chemicals are used, sealing the deck with a low permeability overlay is often a cost effective long term preventive maintenance strategy.
- **Moderate Corrosion Potential** – Doing nothing is not advised for moderate corrosion potential decks with identified delaminations. The delamination may provide a pathway for accelerated deterioration.
- **High Corrosion Potential** – Doing nothing is not advised for high corrosion potential decks with identified delaminations. The delamination may provide a pathway for accelerated deterioration.

Repair

This action class includes spall and delamination repair methods for full and partial depth repairs that are described in Chapter 7. Repairs may be done with a vast number of concrete patch products. Usually these are rapid setting concrete products with high bond strengths. For moderate and high corrosion potential decks, sacrificial anodes should be placed within the patch area to increase the life of the repair.

- **Low Corrosion Potential** decks may warrant spall and delamination repair without further action if delaminations are less than 10% of the total deck area. The low probability of corrosion in these decks wouldn't typically warrant the expense of sealing or overlays. However, in areas where deicing chemicals are used, sealing the deck with a low permeability overlay is often a cost effective long term preventative maintenance strategy.
- **Moderate Corrosion Potential** decks may warrant repair of spalls and delaminations without further action if combined area is less than 5% of the total deck area.
- **High Corrosion Potential** decks should always include an additional protection action (sealing overlay, sacrificial anode or cathodic protection) when delaminations are repaired.

Repair and Seal

This action class involves repairing all spalls and delaminations using techniques described in Chapter 7 and as described under the repair action above. In addition, the combination of cracking severity and spalls/delaminated area warrants sealing the deck from the ingress of water and salts. Sealing the deck is typically accomplished using a penetrating sealer or water proofing treatments, however low permeability overlays will also seal the deck. See Section 20.4.5 for more information on deck sealers.

Overlay

This action class includes a vast number of overlay products. The recommendation for an overlay is to span over and seal larger or densely spaced deck cracks and to provide a low permeability barrier that will limit the amount of water and salts that penetrate into the concrete matrix. Overlays are also effective treatments for rutting, abrasion, and scaling that may have been observed during the visual inspection. Common overlays that will seal include asphalt with a membrane seal, thin bonded polymer overlays, modified concrete overlays and polyester concretes. The selection between overlays that will provide the corrosion protection is typically governed by a variety of factors that may include load capacity, profile grade, joint type, rail height, agency experience and how quickly the overlay can handle traffic. See Section 20.4.5 for more information on deck overlay products.

Rehabilitate

This action class is recommended when the spalled and delaminated deck area exceeds 10% of the total bridge deck area. The rehabilitate action class includes several methods to remove varying depths of concrete, removing rebar corrosion and patching the areas disturbed. Chapter 7 describes the saw cutting, chipping and sand blasting method commonly used for smaller scattered areas and hydrodemolition methods typically used for larger areas. If a deck has experienced this level of spalls and delaminations, a complete deck overlay with a low permeability overlay should be considered to prevent future corrosion damage in the deck. An overlay after rehabilitation will also provide a smooth riding surface on the deck instead of a patchwork of repaired areas (as discussed in Section 20.4.5). Alternatively, an active cathodic protection system could be installed to prevent the future corrosion, although this will not address the ride quality.

Replace

This action class is for complete removal and replacement of the concrete deck. Replacement of the deck may be warranted as the percentage of spalls and delamination increases. Agencies typically use a spalling and delamination percentage of 30 - 50% of the total deck area as the triggering threshold for replacement in lieu of rehabilitation. The presence of exposed and corroded bottom mat reinforcing over significant areas also tips the decision toward deck replacement. Material defects such as ASR or other deterioration that causes a loss of strength in the concrete may also trigger replacement. If the delaminations and spalls are extensive and very deep, the concrete removal process for rehabilitation may require shoring of the bridge during the repair. In these cases, replacement may be the preferred alternative because the cost of rehabilitation can approach replacement costs.

20.4.5 Common Deck Seals and Overlay Products

Many products are commercially available for sealing and overlaying bridge decks. The selection among the available deck sealing products may be governed by attributes such as viscosity, modulus of elasticity, gel times, time until traffic can ride on the treated deck, and temperature at the time of application. Low viscosity sealers can penetrate deeply into fine cracks and higher viscosity sealers are better suited for larger crack widths. High modulus sealers are typically very stiff, while low modulus sealers are flexible.

When to Call the Engineer

Call the Engineer prior to installing or increasing the thickness of overlays.

For overlays, the selection may be governed by cost, thickness of overlay, time required before traffic can use the overlay, and the permeability of the overlay. The additional dead load on the bridge, and the required minimum height of bridge rails, are considerations when determining the allowable thickness of overlays. An engineering review should be conducted prior to installing or increasing the thickness of overlays. Table 20.10 and Table 20.11 present the attributes of common deck seals and overlays, respectively.

Table 20.10 Common Deck Sealing Products

Seal Type	Viscosity (Centipoise)	Modulus of Elasticity	Expected Life (Years)	Time-to Traffic-Use (Hours)	Cost Range (Dollars per Square Foot)
Low Modulus Low Viscosity	≤ 50	Low	5 - 10	2 - 6	0.20 - 3
Low Modulus Super Low Viscosity	> 50	Low	5 - 10	2 - 6	0.20 - 3
High Modulus Low Viscosity	≤ 50	High	5 - 10	4 - 8	0.20 - 3
High Modulus Super Low Viscosity	> 50	High	5 - 10	2 - 6	0.20 - 3

Table 20.11 Common Deck Overlay Products

Overlay Type	Thickness (Inches)	Permeability	Expected Life (Years)	Time-to Traffic-Use (Hours)[6]	Relative Cost[7]
Asphalt w/o Membrane	2 – 12[1,2]	Variable, but generally high	10 - 15	2 - 6	$ - $$
Asphalt with Membrane	3 – 12[1,2,3]	Zero[4]	10 - 20[5]	2 - 6	$$
Thin Bonded Polymers	0.125 - 0.5	Zero[4]	5 - 15	2 - 6	$ - $$$
Modified Concretes	1 - 12	Very Low	20 - 30	4	$$$ - $$$$
Polyester Concretes	0.75 - 12	Very Low	20 - 30	2	$$$ - $$$$

[1] *Asphalt thicknesses over 4 inches are not recommended on bridges due to the weight of the asphalt. The live load capacity of a bridge is decreased by the weight of an overlay.*

[2] *Asphalt overlays provide a protective wearing surface, but tend to trap moisture and chlorides. Asphalt overlays without waterproofing membranes are not recommended for bridge decks where deicing chemicals are used.*

[3] *Three inch minimum is recommended to prevent disturbing membrane during asphalt resurfacing that includes rotomilling and replacing the asphalt to the original thickness.*

[4] *The sealing material is impermeable where there are no imperfections due to installation, aging, or wear.*

[5] *Waterproofing membranes are known to last over 30 years when not disturbed. However, the asphalt typically needs resurfacing every 10 to 20 years to maintain the wearing surface and protect the membrane. It is important to not disturb the membrane during resurfacing operations.*

[6] *Time-to Traffic Use is the time after the last operation has been completed before traffic can be restored.*

[7] *All costs are for deck preparation, furnishing of material and placement.*

20.4.6 Strength Evaluation

Concrete deck strength should be tested during construction to verify that the bridge deck being built meets the design specifications. In rare cases, the deck concrete may lose strength over time due to material deterioration. The most common cause of strength loss of existing bridge decks is ASR. Table 20.12 provides a decision matrix for evaluating strength and ASR results for reinforced concrete bridge decks.

Table 20.12 Concrete Compressive Strength Evaluation

ASR Present?	Concrete Compressive Strength ≥ Design Strength	Concrete Compressive Strength < Design Strength but Acceptable for Applied Loads	Concrete Compressive Strength < Applied Loads (Unacceptable)
ASR Not Present	Do Nothing	Do Nothing	Replace Deck
ASR Potential in Aggregate	Treat ASR	Treat Deck	Replace Deck
ASR Active	Treat ASR	Treat ASR OR Replace Deck	Replace Deck

Do Nothing - If ASR cannot be confirmed by lab testing and the strength of the concrete is adequate for the loads it carries, it is appropriate to do nothing.

Treat ASR - The appropriate treatment could involve penetrating deck sealers, such as high molecular weight methacrylate or protection methods, that would typically involve the application of a low permeability or impervious overlay to limit the moisture flow into the concrete matrix. If the deck can be kept dry, the ASR will be denied the moisture necessary to further react.

Replace Deck - If the concrete strength is found to be inadequate, there are two possible corrective strategies: deck replacement or the construction of a reinforced deck on top of the existing deck.

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Chapter 22 - Bridge Deck Decision Aid Matrices

22.1 Decision Aid Introduction

This chapter presents decision aid matrices that can be used to quickly look up common defects and the feasible actions available to address the defects. Each feasible action references a previous chapter where the detailed repair procedure can be found. Preventive maintenance activities with recurrence intervals are listed in a separate table for each deck type.

22.2 Reinforced Concrete Deck Decision Aid Matrices

22.2.1 Common Reinforced Concrete Defects and Treatments

Table 22.1 Reinforced Concrete Deck/Slab Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Cracking	• Crack Chase - Use penetrating sealer at individual crack locations. (Sections 5.8.1, 7.2.2, 7.2.4) • Seal - Flood coat the deck with penetrating crack sealer (less than 1/8 inch ide cracks). (Sections 7.2.2, 7.2.4, 20.4.4) • Inject - Epoxy Inject larger isolated cracks (1/8 to 1/4 inches wide). (Sections 7.2.2, 7.2.4) Contact an engineer if cracks exceed 1/4 inch wide. • Overlay - Apply asphalt with waterproofing membrane, thin bonded polymer or concrete overlay. (Sections 7.2.3, 7.2.4, 20.4.4)
Delaminations and Spalling	• Patch - Remove unsound concrete, blast clean reinforcing and patch. (Sections 7.2.4, 20.4.4) • Patch and Protect - Remove unsound, blast clean reinforcing, install galvanic anodes and patch. (Sections 7.2.4, 13.1.6, 20.4.4) • Patch Seal and Overlay - Remove unsound concrete, blast clean reinforcing, patch and seal or overlay. (Sections 7.2.4, 20.4.4)
Abrasion	• Overlay - Apply asphalt with waterproofing membrane, thin bonded polymer or concrete overlay. (Sections 7.2.4, 20.4.4)
Rutting	• Overlay - Apply asphalt with waterproofing membrane, thin bonded polymer or concrete overlay. (Sections 7.2.4, 20.4.4)
Scaling	• Overlay - Apply asphalt with waterproofing membrane, thin bonded polymer or concrete overlay. See Chapters (Sections 7.2.4, 20.4.4)
Soffit Efflorescence /Rust	• Sealer or Thin Bonded Polymer Overlay - Minimize moisture in the concrete by applying flood coated deck seal or overlay. (Sections 7.2.4, 20.4.4)
Lack of Rebar Cover	• Overlay - Apply asphalt with waterproofing membrane , thin bonded polymer or concrete overlay. (Sections 7.2.4, 20.4.4)
ASR	• Seal or Overlay - Minimize concrete moisture by applying flood coated deck seal or overlay. (Sections 7.2.4, 20.4.4, 20.4.6) • Repair or Replace - Remove and replace poor quality concrete. (Sections 7.2.4, 20.4.4, 20.4.6)
Lack of Concrete Strength	• Overlay - Construct a reinforced concrete deck on top of the existing deck. (Sections 20.4.4, 20.4.6) • Replace deck. (Sections 20.4.4, 20.4.6)
Loss of Surface Friction	• Roughen Surface - Grind or steel shot the deck surface to roughen. • Saw cut grooves in deck. • Overlay - Apply asphalt with waterproofing membrane, thin bonded polymer or concrete overlay. (Section 7.2.4)
Impact Damage	• Repair - Remove unsound concrete, blast reinforcing clean of any corrosion and patch the damaged area. (Section 7.2.4)

22.2.2 Common Reinforced Concrete Preventive Maintenance Activities

Table 22.2 Reinforced Concrete Deck/Slab Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Sweeping, Cleaning and Pressure Washing, including maintaining and cleaning any deck drains	0.5 - 1
Apply Waterproofing Sealer to prevent water intrusion into concrete in good condition (Silane/Siloxanes)	3 - 5
Applying Penetrating Crack Sealer to widespread finely cracked concrete (Epoxy or Methacrylate, etc.)	5 - 10
Applying Crack Filler into concrete with discrete larger width cracks (Healer/Sealer)	5 - 10
Applying Thin Bonded Polymer Overlay	10 - 20
Applying Rigid Concrete Overlays	15 - 30
Applying Bituminous Overlay (asphalt with waterproofing membrane)	10 - 15
Application of Impressed Current Cathodic Protection System	One time / Ongoing Maintenance
Installation of Galvanic Anodes for Corrosion protection	10-20

22.3 Steel Deck Decision Aid Matrices

22.3.1 Common Steel Deck Defects and Treatments

Table 22.3 Steel Deck Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Steel Corrosion	Repair or replace steel/connection as needed, blast clean and apply protective coating. (Sections 7.3.2, 7.3.3)
Cracked / Broken Fastener	Retrofit or replace steel, blast clean and apply protective coating. (Section 7.3.3)
Bent/Buckled Steel	Repair or replace steel, clean and apply protective coating. (Section 7.3.3)
Loss of Surface Friction	Weld studs on open grid decks or apply overlay to closed decks. (Sections 7.3.3)
Impact Damage	Repair or replace damaged steel, blast clean and apply protective coating. (Section 7.3.3)

22.3.2 Common Steel Deck Preventive Maintenance Activities

Table 22.4 Steel Deck Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Sweeping, Cleaning and Pressure Washing	0.5 - 1
Clean and Paint or Metalizing	10 - 20
Cathodic Protection (Sacrificial Anodes)	10 - 20

22.4 Timber Deck Decision Aid Matrices

22.4.1 Common Timber Defects and Treatments

Table 22.5 Timber Deck and Slab Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Checks	Apply sealant, preservative or protective coating. (Sections 7.4.2, 7.4.4)
Splits	Apply sealant, preservative or protective coating. (Sections 7.4.2, 7.4.4)
Decay	Apply preservative or protective coating. Replace timbers members as needed. (Sections 7.4.2, 7.4.4)
Crushing	Remove decayed section and supplement or replace with new timbers spliced or scabbed onto sound material. (Section 7.4.4)
Wear/Abrasion	Replace timber runners or apply overlay. (Sections 7.4.3, 7.4.4)
Pest Infestation	Treat with fumigants if timber is salvageable or replace timber. (Section 7.4.2, 7.4.4)
Nail Laminated Deck Delamination	Stress laminate the delaminated length.
Loose/Missing Fasteners	Tighten or replace fasteners and apply preservative. Consider bolted fasteners with washers when possible in lieu of lag bolts.

22.4.2 Common Timber Preventive Maintenance Activities

Table 22.6 Timber Deck/Slab Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Sweeping, Cleaning and Pressure Washing	0.5 - 1
Secure Loose or Replace Missing Fasteners	As Needed
Applying Preservatives	3 - 5
Treat with Fumigants	10 - 15
Replace Damaged or Deteriorated Timbers	As Needed
Applying Bituminous Overlay (Asphalt) with Membrane	10 - 15

22.5 Fiber Reinforced Polymer (FRP) Deck Decision Matrices

22.5.1 Common FRP Defects and Treatments

Table 22.7 Fiber Reinforced Polymer (FRP) Deck/Slab Decision Aid Matrix

Common Deficiency	Feasible Treatments
Loose Fasteners	Tighten loose or replace broken or missing fasteners
Delaminations	Remove and replace delaminated FRP area using wet lay-up of new FRP material compatible with the existing. Consult engineer.

22.5.2 Common FRP Preventive Maintenance Activities

Table 22.8 FRP Deck Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Applying Thin Bonded Polymer Overlay	10 - 20
Applying Rigid Concrete Overlays	20 - 30
Applying Bituminous Overlay (Asphalt with membrane)	15 - 20

22.6 Chapter 22 Reference List

1. Chapters 7 and 20, this manual.
2. FHWA. *Bridge Inspector's Reference Manual (BIRM)*. Publication No. FHWA NHI 12-049. Washington D.C.: United States Department of Transportation, October 2002, Revised December 2006, Revised February 2012.
3. NCHRP. *Guidelines for Selection of Bridge Deck Overlays, Sealers and Treatments*. NCHRP Project 20-07, Task 234, http://maintenance.transportation.org/Documents/Final%20Report%2020-07%20Task%20234.pdf

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Chapter 23 - Superstructure Decision Aid Matrices

23.1 Decision Aid Introduction

This chapter presents decision aid matrices that can be used to quickly look up common defects and the feasible actions available to address the defects. Feasible actions reference previous chapters where detailed discussion or repair procedures can be found. Preventive maintenance activities with recurrence intervals are listed in separate tables for each superstructure material and related superstructure components.

23.2 Concrete Superstructure Decision Aid Matrices

23.2.1 Common Concrete Defects and Treatments

Table 23.1 Concrete Superstructure Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Cracking	• Seal/Repair Crack – Seal hairline superstructure cracks (epoxy, etc.). • Inject – Epoxy Inject larger superstructure cracks (0.025 inch or wider). (Sections 11.1.4, 11.4.3) • Seal Slab Cracks – Seal top surface cracks in slab superstructures. (Section 7.2.2) • Epoxy Inject – Inject soffit cracking in slab superstructures. (Section 11.4.3)
Delaminations	• Patch and Protect – Remove unsound concrete 1 inch beyond reinforcing steel, blast clean reinforcing, install galvanic anodes and patch. (Sections 7.2.4, 11.4.2, 13.1.6)
Spalling	• Patch – Shallow Patching – Remove unsound concrete 1 inch beyond reinforcing steel, blast clean reinforcing and patch. (Section 11.4.2) • Repair – Extensive Spall Repair – Check with an engineer before removing concrete. Remove unsound concrete, blast clean reinforcing, recast concrete or shotcrete. (Sections 11.4.2, 11.4.2) • Repair and Protect – Galvanic anodes or impressed current (active) cathodic protection can be paired with spall repairs for enhanced corrosion protection. (Sections 7.2.4, 13.1.6)
Reinforcing Steel Repair	• Supplement Rebar – Splicing in new rebar due to corrosion or damage. (Section 11.4.6) • Replace Rebar – Replace corroded or damaged rebar using mechanical splices. (Section 11.4.6)
Rebar Corrosion	• Repair – Remove loose concrete, blast clean steel, supplement steel as needed, patch and seal. (Sections 11.4.2, 11.4.6) • Protect – Install Galvanic Anode Corrosion Protection. (Sections 7.2.4, 13.1.6) • Protect – Install impressed current Cathodic Protection System. (Section 13.1.6)
Prestressing Steel Repair	• Splice Prestressed Strands – Consult an engineer. Remove concrete away from the damaged strand, cut existing strand, install and torque coupler, recast concrete. (Section 11.4.6)
Beam End Damage	• Beam End Repair – Consult an engineer. Jack up girder, remove unsound concrete, blast clean reinforcing, supplement steel as needed and patch. (Section 11.4.4)
Strengthening with FRP	• Strengthen – Apply fiber reinforced polymer strips, sheets or wraps. (Section 11.4.5)
Impact Damage	• Inject Concrete Cracks. (Section 11.4.3) • Repair Reinforcing. (Section 11.4.6) • Repair Damaged Concrete. (Section 11.4.2) • Strengthen Damaged Area – Apply Fiber Reinforced Polymer Strips or Sheets. (Section 11.4.5)

23.2.2 Common Concrete Preventive Maintenance Activities

Table 23.2 Concrete Superstructure Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Cleaning*	0.5 - 1
Debris Removal*	0.5 - 1
Pressure Washing**	1 - 2
Applying Penetrating Crack Sealer	5 - 10
Bird Netting / Deterrent	One time / Maintenance as needed
Waterproofing Treatments	10 - 15
Application of Impressed Current Cathodic Protection System	One time / Ongoing Maintenance
Installation of Galvanic Anodes for Corrosion protection	10 - 20

*Cleaning and debris removal can be performed on a cyclical basis or as needed depending on the source and accumulation frequency.

**Pressure washing is most beneficial where the members are exposed to saturation from salt contaminated water. Common locations include superstructure areas under joints, splash or spray zones, and areas with bird debris.

23.3 Steel Superstructure Decision Aid Matrices

23.3.1 Common Steel Superstructure Defects and Treatments

Table 23.3 Steel Superstructure Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Steel Corrosion	• Repair – Repair steel/connection as needed, blast clean and apply protective coating. (Section 11.5.2) • Replace Steel – Consult Engineer before starting work. Replace corroded section. (Section 11.5.2)
Fatigue Cracking	• Grind – Consult Engineer for primary structural members. Grind out small cracks in welds. (Section 11.5.3) • Arrest fatigue – Consult Engineer for primary structural members. Arrest cracks, retrofit or replace steel, blast clean and apply protective coating. (Section 11.5.3) • Supplement Member – Consult Engineer for primary structural members. Supplement cracked section with doubler plates. (Section 11.5.3)
Bent/Buckled Steel	• Straighten Member – Consult Engineer for primary structural members. (Section 11.5.4) • Install Stiffening Members – Consult Engineer. • Replace Member – Consult Engineer.
Loose, Missing or Broken Fasteners	• Tighten bolts as needed. • Replacing rivets or bolts. (Section 11.5.2)
Gusset Plate Distortion	• Install stiffening angles, consult an engineer. (Section 12.4.3)
Gusset Plate Repair	• Stiffen Gusset Plate, consult an engineer. (Section 12.4.3) • Supplement/Strengthen – Install Reinforcing Plate as needed for section loss or damage, consult an engineer. (Section 12.4.3)
Improper Seismic Restrainer Tension	• Adjust Seismic Cable Restrainer. Consult an engineer for required cable sag/tension requirements. (Section 12.6.3)
Broken Seismic Cable Restrainer	• Replace – Consult an engineer for proper cable replacement specifications.
Impact Damage	• Repair – Straighten damaged member, prepare surface and apply coating. Consult Engineer for primary structural members. (Section 11.5.4) • Replace damaged steel – Consult an Engineer.

Table 23.4 Suspension and Cable Stayed Bridge Decision Aid Matrix

Common Deficiency	**Feasible Treatments** Refer to Section 12.5 for additional information
Coating System / Wrapping Failure	• Caulk cable bands, saddles and other connection points. • Repaint or remove and replace cable wrapping.
Cable Corrosion	• Apply corrosion inhibitor to strands. Consult an engineer before starting work.
Cable Pitting or Section Loss	• Protect/Repair – Main suspension cables and cable stays – Consult an Engineer. • Repair or Replace coating – Suspender cables – Consult an engineer. • Replace suspender cables – Consult an engineer.
Fatigue Cracking	• Repair – Broken Strands in main suspension cables and cable stays – consult an engineer. • Repair – Broken strands in suspender cables – Consult an engineer. • Repair or replace coating – Suspender cables – Consult an engineer.
Loose, Missing or Broken Fasteners	• Secure – Tighten bolts as needed in cable bands and saddles or other inspection safety cables. • Repair – Broken or missing anchors of main cables, cable stays or suspender cables – Consult an engineer.
Secure Suspension Hand Cables	• Tighten fasteners /Adjust tension. • Replace cables with section loss.
Broken Cable Strands	• See “Fatigue Cracking” above.
Tighten Cable Band	• Tighten cable band fasteners – Consult an engineer.
Cable Compaction	• Compact main cable as needed – Consult an engineer.
Excessive Cable Vibrations	• Dampen cable vibrations – Consult an engineer.

23.3.2 Common Steel Superstructure Preventive Maintenance Activities

Table 23.5 Steel Superstructure Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Cleaning*	0.5 - 1
Debris Removal*	0.5 - 1
Pressure Washing**	1 - 2
Install Bird Netting / Deterrent	One Time/Maintenance as Needed
Painting or Metalizing (environment dependent)***	10 - 30
Corrosion Protection (Galvanic)	10 - 20
Apply Cable Corrosion Inhibitor	Upon Observed Corrosion
Cable Vibration Control/Reduction	Upon Observed Vibration
Cable Band Caulking	2 - 3
Cable Rewrapping	Upon Breakdown/Corrosion of Wrapping
Cable Dehumidification	Maintain Automatic Humidity Controlled Systems
Adjusting Seismic Restrainer Cable Tension	As Needed / Post Event
Replacing Missing/Loose Seismic Restrainer Fasteners	As Needed

*Cleaning and debris removal can be performed on a cyclical basis or as needed depending on the source and accumulation frequency.

**Pressure washing is most beneficial where the members are exposed to saturation from salt contaminated water. Common locations include superstructure areas under joints, splash or spray zones, and areas subject to birds.

***Painting includes spot painting, zone painting, and repainting entire bridge. See Section 18.3.

23.4 Timber Superstructure Decision Aid Matrices

23.4.1 Common Timber Defects and Treatments

Table 23.6 Timber Superstructure Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Checks	• Seal – Apply sealant, preservative or protective coating. (Section 11.3.3) • Seal – Fill checks with epoxy. (Section 11.3.4)
Splits	• Seal – Apply sealant, preservative or protective coating. (Section 11.3.3) • Repair – Treat splits with epoxy. (Section 11.3.4) • Repair by clamping the split section. (Sections 11.3.5, 11.6.1) • Repair by stitching the split section. (Section 11.3.5) • Supplement member – Scab a new member over the split section. (Section 11.6.2)
Decay/Crushing	• Preserve – Apply preservative or protective coating for surface decay if timber is salvageable. (Section 11.3.3) • Preserve – Apply fumigants to treat internal decay and fungi if timber is salvageable. (Section 11.3.3) • Repair – Remove decayed/adjacent wood and fill void with epoxy. (Section 11.3.4) • Supplement – Remove decayed section and supplement or replace with new timbers spliced or scabbed onto sound material. (Section 11.6.2) • Replace – Replace member where required or less costly than preserving, repairing, or supplementing. (Section 11.6.4) • Repair with Epoxy (Section 11.3.4)
Pest Infestation	• Treat with fumigants if timber is salvageable. (Section 11.3.3)
Loose/Missing Fasteners	• Tighten or replace fasteners and apply preservative. Consider bolts with washers when possible in lieu of lag bolts.
Inadequate Strength	• Scab steel plates onto the timber. (Section 11.6.2) • Post tension the timber member. (Section 11.6.3) • Replace – Replace member with least costly alternative. (Section 11.6.4)
New Beam Required	• Replace timber member. (Section 11.6.4)

23.4.2 Common Timber Preventive Maintenance Activities

Table 23.7 Timber Superstructure Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Cleaning and Pressure Washing*	0.5 - 1
Secure Loose or Replace Missing Fasteners	As Needed
Apply Waterproofing Seal	2 - 10
Applying Preservatives	3 - 5
Treat with Fumigants	10

*Cleaning and pressure washing can be performed on a cyclical basis or as needed depending on the source and the debris accumulation frequency. Pressure washing should be performed at pressures low enough to avoid damage to the surface of the wood. If pressure washing damages surface of timber, other methods such as flooding should be considered.

23.5 Bearing Decision Matrices

23.5.1 Common Bearing Defects and Treatments

Table 23.8 Bearing Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Poor Alignment/ Rotation	• Reset the bearings. (Section 12.3.3) • Install bearing restraints as needed. (Section 12.3.5)
Corrosion/ Section Loss	• Clean and apply protective coating. (Section 12.3.6)
Loose, Broken or Missing Fasteners	• Tighten loose or replace missing fasteners – As needed. • Replace anchor bolts. (Section 12.3.4)
Deteriorated Seat Concrete	• Remove unsound concrete and patch. (Section 13.4.2)
Sheared/Crushed Elastomeric Pads	• Replace bearing pads. (Section 12.3.1)
Deteriorated Condition or Function	• Replace when unable to repair. (Section 12.3.1)

23.5.2 Common Bearing Preventive Maintenance Activities

Table 23.9 Bearing Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Pressure Washing**	0.5 - 1
Cleaning / Debris Removal	1 - 2
Lubricating Bearing (if needed)	1 - 2
Painting or Metalizing	10 - 20
Anchor Bolt Tightening	As Needed
Securing Fasteners	As Needed

*Cleaning and debris removal can be performed on a cyclical basis or as needed depending on the source and accumulation frequency.

** Pressure washing is most beneficial where the members are exposed to saturation from salt contaminated water. Bearings in benign environments, protected from salt contaminated water or protected from debris accumulation may not warrant pressure washing at the noted frequency.

23.6 Chapter 23 Reference List

1. Chapters 11 and 12, this manual.
2. FHWA. *Bridge Inspector's Reference Manual (BIRM)*. Publication No. FHWA NHI 12-049. Washington D.C.: United States Department of Transportation, October 2002, Revised December 2006, Revised February 2012.
3. FHWA. *Primer for the Inspection and Strength Evaluation of Suspension Bridge Cables*. Publication No. FHWA-IF-11-045. Washington D.C.: United States Department of Transportation, May 2012, http://www.fhwa.dot.gov/bridge/pubs/if11045.pdf

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Chapter 24 - Bridge Substructure Decision Aid Matrices

24.1 Decision Aid Introduction

This chapter presents decision aid matrices that can be used to quickly look up common defects and the feasible actions available to address the defects. Feasible actions reference previous chapters where detailed discussion or repair procedures can be found. Preventive maintenance activities with recurrence intervals are listed in separate tables for each superstructure material and related superstructure components.

24.2 Reinforced Concrete Substructure Decision Aid Matrix

24.2.1 Common Reinforced Concrete Condition Based Defects and Treatments

Table 24.1 Concrete Substructure Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Corrosion	• Apply corrosion inhibitor to concrete prior to repairing spalls and delamination. (Section 5.2.4) • Application of Impressed Current Cathodic Protection System. (Section 13.1.6) • Installation of Galvanic Anodes. (Section 13.1.6) • Activated arc spray zinc. (Section 13.1.6) • Apply Electrochemical Techniques. (Section 13.1.7)
Cracking: Non-Working Cracks	• Inject with epoxy or urethane based product – Drill holes along crack, clean and dry crack, install ports, seal cracks with an epoxy mastic before injecting epoxy. (Sections 5.8.3, 13.4.5) • FRP Wrap for containment. (Section 13.4.10)
Cracking: Working Cracks	• Use flexible sealant such as an asphalt crack sealer, rout crack, clean and dry substrate, fill with flexible sealant. (Section 5.8.1)
Concrete Deterioration: Non-Structural Repair	• Form and Pour (Recasting) with concrete tied into rebar, patch with cementitious or polymer-based material. (Sections 5.7, 13.4.1) • Consider Shotcrete. (Section 13.4.4)
Concrete Deterioration: Structural Repair	• Check with an engineer before removing concrete, structural lift and shore as per approved procedure, provide supplemental support, drill & grout new rebar into sound concrete, repair with material of appropriate compressive strength. (Sections 13.4.2, 13.7)
Delaminations & Spalling	• Remove unsound concrete, blast clean substrate and rebar and patch. (Sections 5.6, 5.7) • Galvanic anodes or impressed current cathodic protection can be paired with spall repairs for enhanced protection against corrosion. (Section 13.1.6)
Lack of Rebar Cover	• Apply penetrating sealers or coatings. (Sections 13.1.4, 13.1.5) • FRP wrap for containment. (Section 13.4.10)
Lack of Concrete Strength	• Shore and support. (Section 13.7) • Apply reinforced concrete jacket tied into existing or FRP. (Section 13.4.3)
Lack of Reinforcing Strength	• External strengthening with FRP. (Section 13.4.10) • External reinforcing such as post-tensioning. (Section 13.4.11)

Chloride Contamination	• Apply penetrating sealers or coatings. (Sections 13.1.4, 13.1.5)
	• Install cathodic protection. (Section 13.1.6)
	  ▪ Precast elements: apply zinc mesh and jacket with concrete
	  ▪ Cast-in-place: apply titanium mesh and jacket with concrete
	  ▪ Marine: thermal spray zinc or install zinc hydrogel anode
	• Apply Electrochemical Extraction. (Section 13.1.7)

Impact Damage	• Perform concrete repairs. (Section 5.7) • FRP wrap for strength. (Section 13.4.10)
Uniform Settlement (Simple Span Type Structures)	• Reset joints and bearings and monitor. (Section 13.4.7)
Differential Settlement	• Re-establish sound foundation per approved plan. (Section 13.4.7)
Scour	• Apply bank stabilization techniques, riprap, gabions and mattress. (Section 15.3.1) • Tremie Concrete. (Section 15.3.2) • Grout Bags. (Section 15.3.3) • Sheet Piling. (Section 15.3.4) • Articulated Block. (Section 15.3.5) • Biotechnical, not covered in this manual, see HEC-23 Chapter 6
Undermining	• Repair using concrete tremies, pumped, bagged or prepacked. (Section 13.4.8)

24.2.2 Common Reinforced Concrete Preventive Maintenance Activities

Table 24.2 Concrete Substructure Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Sweeping, Cleaning, and Pressure Washing*	0.5 - 2
Applying Penetrating Crack Sealer	8 - 10
Applying Flexible Crack Filler	2 - 4
Debris Removal	As needed
Waterway Control	As needed
Restore Drainage/ Flushing	0.5 - 2
Erosion Control	As needed
Remove Vegetation	0.5 - 2
Replace Riprap	As needed

*Cleaning and debris removal can be performed on a cyclical basis or as needed depending on the source and accumulation frequency. Pressure washing is most beneficial where the members are exposed to saturation from salt contaminated water. Common locations include superstructure areas under joints, areas with bird debris, etc.

24.3 Steel Substructure Decision Aid Matrix

24.3.1 Common Steel Substructure Condition Based Defects and Treatments

Table 24.3 Steel Substructure Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Steel Corrosion: Coating Issues	• Apply Coating. (Section 18.6)
Steel Corrosion: Electrical Issues	• Apply cathodic protection – Sacrificial anodes, activated arc spray zinc. (Section 13.1.6)
Steel Corrosion: Section Loss	• Repair as needed (welded, bolted plates). (Section 13.5.2) • Replace Element(s). (Sections 11.5.2, 13.5.3)
Fatigue Cracking	• Arrest the crack, retrofit or replace section, clean and apply protective coating. (Section 11.5.3)
Bent/Buckled Steel	• Heat Straighten if possible. (Section 11.5.4) • Repair (welded, bolted plates) . (Sections 11.5.2, 13.5.3) • Repair area and/provide supplemental support. (Section 13.5.3)
Crevice Corrosion	• Clean and recoat. (Sections 13.2.2, 18.6)
Impact Damage	• Provide supplemental support if necessary. (Section 13.5.5) • Heat straighten or replace member(s). (Section 11.5.4) • Shoring and Temporary Supports. (Section 13.7) • Fiber Reinforced Polymer Repairs. (Section 13.4.10)

24.3.2 Common Steel Substructure Preventive Maintenance Activities

Table 24.4 Steel Substructure Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Sweeping, Cleaning, and Pressure Washing*	0.5 - 2
Spot Paint	8 - 10
Cathodic Protection (Sacrificial Anodes)	2 - 4
Metalizing / Overcoating	As needed
Galvanizing	As needed
Debris Removal	0.5 - 2
Waterway Control	As needed
Restore Drainage / Flushing	0.5 - 2
Erosion Control	As needed
Remove Vegetation	0.5 - 2

*Cleaning and debris removal can be performed on a cyclical basis or as needed depending on the source and accumulation frequency. Pressure washing is most beneficial where the members are exposed to saturation from salt contaminated water. Common locations include superstructure areas under joints, areas with bird debris etc.

24.4 Timber Substructure Decision Aid Matrix

24.4.1 Common Timber Defects and Treatments

Timber repairs involve repair of individual timber piles and timber sway bracing. Pile posting, pile restoration and pile shimming all incorporate a new piece of treated timber in the repair. These methods are cost effective, but will be subjected to the same deterioration. Concrete jacketing, pile augmentation and PVC wrapping are methods which leave the existing pile in its deteriorated state, but replace the section loss with concrete and usually provide a watertight seal. These three methods are more expensive, but provide a level of protection against future deterioration.

Table 24.5 Timber Substructure Condition Based Decision Aid Matrix

Common Deficiency	Feasible Treatments
Checks	• Apply protective sealant, epoxy injection, preservative, or protective coating. (Section 13.3.3)
Splits	• Scab/splice with new treated timbers. (Section 13.6.2) • Epoxy inject the split. (Section 11.3.4) • Provide supplemental support and replace elements(s). (Section 13.7) • Apply protective sealants, preservatives or coatings. (Section 13.3.3) • Reinforce with Fiber Reinforced Polymer (FRP) wraps/strips. (Section 13.6.7)
Decay	• Provide supplemental support, if necessary. (Section 13.7) • Remove decayed area or use an in-place treatment (diffusible, non-diffusible, or a fumigant). (Section 13.3.3) • Apply protective sealant, preservative, or protective coating. (Section 13.3.3) • Scab/splice with new timbers. (Section 13.6.2) • Replace elements(s)
Crushing	• Provide supplemental support, if necessary. (Section 13.7) • Replace damaged sections and supplement or replace with new timbers spliced or scabbed onto sound material. (Section 13.6.2) • Apply protective coating or preservative. (Section 13.3.3) • Apply preservative to all cuts and bolt holes. (Section 13.3.3)
Wear/Abrasion	• Remove debris • Provide supplemental support and replace worn sections with treated lumber. (Section 13.7) • Scab/splice with new timbers. (Section 13.6.2) • Protect with PVC or fiberglass liner. (Section 13.6.1)
Pest Infestation	• Treat with a diffusible or a fumigant if timber is salvageable (Section 13.3.3) or replace timber. • Apply protective coating or preservative. (Section 13.3.3) • Scab/splice with new timbers. (Section 13.6.2)
Loose/Missing Fasteners	• Replace missing hardware with metals compatible with preservative • Treat cuts and bolts holes with preservative. (Section 13.3.3)

24.4.2 Common Timber Preventive Maintenance Activities

Table 24.6 Timber Substructure Cyclical Preventive Maintenance Activities

Common Activities	Typical Frequency (Years)
Sweeping, Cleaning, and Pressure Washing*	0.5 - 2
Secure Loose or Replace Missing Fasteners	As needed
Applying Preservatives	3 - 5 for above ground 1 - 3 for timber in ground contact
Treat in-place with either a diffusible, a non-diffusible, or a fumigant	10 - 15
Replace Damaged or Deteriorated Timbers	As needed
Remove Brush & Vegetation	0.5 - 2
Moisture / Drainage Control	0.5 - 2 See Chapter 13.3.2

*Cleaning and debris removal can be performed on a cyclical basis or as needed depending on the source and accumulation frequency. Pressure washing is most beneficial where the members are exposed to saturation from salt contaminated water. Common locations include superstructure areas under joints, areas with bird debris, etc.

24.5 Chapter 24 Reference List

1. Chapters 5, 7, and 13, this manual.
2. FHWA. *Bridge Inspector's Reference Manual (BIRM)*. Publication No. FHWA NHI 12-049. Washington D.C.: United States Department of Transportation, October 2002, Revised December 2006, Revised February 2012.
3. Illinois Department of Transportation. Bureau of Bridges and Structures Division of Highways. *Bridge Element Inspection Manual*, February 2014.
4. Wan, Baolin and Travis McDaniel. *Repair of Deteriorated Bridge Substructures*. Project 0092-11-08, sponsored by the Wisconsin Highway Research Program (WHRP), May 2013, http://wisdotresearch.wi.gov/wp-content/uploads/WisDOT-WHRP-project-0092-11-08-brief.pdf

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Chapter 25 - Culvert Matrix

25.1 Decision Aid Introduction

This chapter presents a decision aid matrix that can be used to quickly look up common defects and the feasible actions available to address the defects. Feasible actions reference previous chapters where detailed discussion or repair procedures can be found.

Table 25.1 Condition Based Decision Making Matrix for Culverts (Part 1)

Common Deficiency	Feasible Treatments
Leaking Joints	• Install internal bands. (Section 14.6.3) • Install internal joint seals. (Section 14.6.3) • Grout injection into soil around leaking joints. (Section 14.6.3) • Construct concrete collar around leaking joints. (Section 14.6.3) *The following options are only likely to be economically feasible if the majority of the joints have failed:* • Slip line culvert with Corrugated Metal Pipe (CMP). (Section 14.6.2.1) • Slip line culvert with polyethylene pipe. (Section 14.6.2.1) • Slip line culvert with smooth interior metal pipe. (Section 14.6.2.1) • Line pipe with inversion liner. (Section 14.6.2.3) • Line pipe with shotcrete/spin cast concrete. (Section 14.6.2.2)
Waterway Opening: Vegetation	• Cut brush/trees. (Section 14.5.1.3) • Clear vegetation and grub roots. (Section 14.5.1.3)
Waterway Opening: Silt/Debris	• Mechanical removal. (Section 14.5.1.2) • Vacuum truck. (Section 14.5.1.2) • Water jet/Sewer jet. (Section 14.5.1.2) • Fire hose flush. (Section 14.5.1.2) • Bucket line. (Section 14.5.1.2)
Inlet/Outlet Scour	• Rip Rap. (Section 14.5.2) • Grout Injection. (Section 14.5.2) (Note: This treatment is only likely to be necessary if scour has advanced beneath, and has undermined the culvert.) • Construct Toe Wall. (Section 14.6.4)
Masonry Culverts: Loss of Mortar	• Repoint. (Sections 14.6.4, 16.3.1)
Masonry Culverts: Cracks	• Repoint. (Sections 14.6.4, 16.3.1) • Replace or Repair Masonry Units. (Sections 16.3.4, 16.3.6)
Bottomless Culverts: Footing Undermined	• Grout injection. (Section 14.5.2) • Footing Repair. (Section 14.6.6)

Table 25.2 Decision Making Matrix for Culverts (Part 2)

Common Deficiency	Feasible Treatments
Barrel: Damaged Invert	• Pave invert with concrete. (Section 14.6.1.1) • Install saddle plates. (Section 14.6.1.2) • Slip line culvert with Corrugated Metal Pipe (CMP). (Section 14.6.2.1) • Slip line culvert with polyethylene pipe. (Section 14.6.2.1) • Slip line culvert with smooth interior metal pipe. (Section 14.6.2.1) • Line pipe with inversion liner. (Section 14.6.2.3) • Line pipe with shotcrete/spin cast concrete. (Section 14.6.2.2)
Barrel: Damaged Coatings	• Repair Coating in accordance with ASTM A780 (not economically feasible if there is general failure of the coatings over a majority of the surface area) • Pave invert with concrete. (Section 14.6.1.1) • Install saddle plates. (Section 14.6.1.2) *The following options are only likely to be appropriate if there are other issues like corrosion or joint failure:* • Slip line culvert with Corrugated Metal Pipe (CMP). (Section 14.6.2.1) • Slip line culvert with polyethylene pipe. (Section 14.6.2.1) • Slip line culvert with smooth interior metal pipe. (Section 14.6.2.1) • Line pipe with inversion liner. (Section 14.6.2.3) • Line pipe with shotcrete/spin cast concrete. (Section 14.6.2.2)
Barrel: Corrosion	• Pave invert with concrete. (Section 14.6.1.1) • Install saddle plates. (Section 14.6.1.2) • Slip line culvert with Corrugated Metal Pipe (CMP). (Section 14.6.2.1) • Slip line culvert with polyethylene pipe. (Section 14.6.2.1) • Slip line culvert with smooth interior metal pipe. (Section 14.6.2.1) • Line pipe with inversion liner. (Section 14.6.2.3) • Line pipe with shotcrete/spin cast concrete. (Section 14.6.2.2)
Barrel: Cracks	• Slip line culvert with Corrugated Metal Pipe (CMP). (Section 14.6.2.1) • Slip line culvert with polyethylene pipe. (Section 14.6.2.1) • Slip line culvert with smooth interior metal pipe. (Section 14.6.2.1) • Line pipe with inversion liner. (Section 14.6.2.3) • Line pipe with shotcrete/spin cast concrete. (Section 14.6.2.2)
Barrel: Deformation	• Line pipe with inversion liner. (Section 14.6.2.3) • Line pipe with shotcrete/spin cast concrete. (Section 14.6.2.2)
Barrel: Concrete Spalls	• Patch Spalls. (Section 5.7.6) • Repair Concrete Surface. (Section 13.4.1) • Slip line culvert with Corrugated Metal Pipe (CMP). (Section 14.6.2.1) • Slip line culvert with polyethylene pipe. (Section 14.6.2.1) • Slip line culvert with smooth interior metal pipe. (Section 14.6.2.1) • Line pipe with inversion liner. (Section 14.6.2.3) • Line pipe with shotcrete/spin cast concrete. (Section 14.6.2.2)

25.2 Chapter 25 Reference List

1. American Society for Testing and Materials. *Standard Practice for Repair of Damaged and Uncoated Areas of Hot-Dip Galvanized Coatings.* ASTM A780, 2009.
2. Chapters 5, 13, 14, and 16, this manual.

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Appendix A - Job-Aid Checklists

A.1 Bridge Cleaning

Description

Remove all salt, dirt, and debris from the deck and supporting members, bearings, pedestals, capbeams, and bridge seats. Unplug and clean the drainage system (scuppers, open joint troughs, and downspouts) on the bridge. Clean debris and vegetation from around the structure and approaches.

Project Objectives

Remove dirt and debris that traps moisture and de-icing salt to prevent corrosion and to enable elements to function properly. Ensure proper drainage on and around the bridge.

Labor Skills

Physical labor

Materials and Equipment

- Clean water
- Equipment
- Water trailer
- Sewer snake
- Long-handle scrapers
- Square-point shovels
- Brush clippers and saw
- High-pressure water pump with hoses, etc.
- 175-CFM air compressor with hoses, etc.
- Ladders and scaffolding
- Brooms
- Personal safety equipment
- Wheel barrow

Procedure

- Coordinate with Agency Environmental Unit and obtain any necessary permits from the State Environmental Agency, US Army Corps of Engineers, or EPA, particularly for acceptable months, to wash at environmentally sensitive or protected streams.
- Prepare the work-zone (e.g., traffic control, environmental protection, other equipment).
- Cut and remove vegetation from around substructures and approaches, minimizing removal to limit habitat loss, erosion, and sedimentation.
- Collect and remove trash, dirt, and other debris on and around the bridge (including underside, supporting elements, and approaches) by sweeping, shoveling, vacuuming, or other suitable methods.
- Loosen dirt and debris with scrapers and brooms as necessary.
- Properly dispose of all collected material. If sand, dirt, or other similar material is

to be disposed of on-site, place it in an upland area from which it cannot enter a stream, water body or wetland.

- Pressure or flood wash the structure, generally beginning at the highest point and working downward, using a water source and pressures that are recommended by the bridge owner or agency. Carefully avoid excessive pressure that may damage paint, grout, or other materials.

Figure A.1 Bridge Washing

- Flush scuppers and drainage system, but prevent sediment and debris from discharging into streams and other surface-water bodies.
- Remove scupper gratings and downspout clean-out plugs to flush and snake trapped debris, as necessary.
- Use caution to control water pressure used in flushing drainage systems.
- Portions of structures with loose paint chips should not be pressure-washed, and care should be taken to collect and properly dispose of any dislodged paint chips

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Consider safety needs for feature crossed and utility lines.
- Identify and avoid pathogen carrying debris (bird droppings). If debris is to be removed, use appropriate PPE and follow latest OSHA/Health Department guidance.

A.2 Sealing Cracks and Joints in Bituminous Wearing Surface

Description

Clean out and seal cracks and joints in bituminous wearing surface to protect the underlying structural deck.

Project Objectives

Minimize or eliminate water and chlorides entering the structure through these cracks and extend the life of the asphalt-cement-concrete wearing surface.

Labor Skills

Physical labor

Materials

- Crack-sealer meeting ASTM D 6690 or polymer-based "healer/sealer"
- Blasting sand
- Equipment
- Crack-sealer heater/melter
- Wand, hoses, and nozzles
- Air compressor with hoses, etc.
- Shovels and brooms
- Concrete saw (crack-chaser)
- Sandblaster
- Personal safety equipment
- Grinder
- Putty knife

Procedure

- Prepare the work-zone (i.e., traffic control, environmental protection, equipment).
- Loosen and shovel off heavy dirt deposits.
- Clean wearing surface by sweeping and/or using compressed air.
- Clean cracks and joints using water or compressed air, and a grinder or putty knife to scrape out larger deposits or old joint material.
- Prepare joint surfaces by sandblasting, sawcutting, or grinding, if necessary.
- Clean all loose dirt and sandblast material generated by joint surface preparation from pavement surface.
- Cracks and joints should be clean and dry before applying sealing material.
- Apply D 6690 crack sealer or Polymer-based material liquid joint sealer to joints and cracks according to manufacturer instructions, allowing adequate time for material to dry.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.

Figure A.2 Applying a Crack Sealer

A.3 Crack Sealing in Portland Cement Concrete Decks

Description

Applying ASTM D 6690 Joint and Crack sealant to cracks in concrete bridge decks. Also, applying epoxy injection and sealers such as HMWM.

Project Objectives

Minimize or eliminate water and chlorides entering the structure through these cracks. Extend the service life of the existing bridge deck.

Labor Skills

Physical labor

Materials

- Crack-sealer meeting ASTM D 6690
- Blasting sand

Equipment

- Crack-sealer heater/melter
- Wand, hoses, and nozzles
- Air compressor with hoses, etc.
- Shovels and brooms
- Walk behind concrete router
- Sandblaster
- Personal safety equipment
- Grinder and putty knife

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Clean deck by sweeping and/or using compressed air.
- Clean cracks and joints using water or compressed air, and a grinder or putty knife to scrape out larger deposits or old joint material.
- Using concrete router, route channel along existing crack to a depth of +/- 1 inch.

Figure A.3 Crack Sealing in Concrete Decks

- Remove debris from routing operation.
- Sandblast routed crack, clean up sandblasting debris.
- Surface must be clean, dry, and temperatures must be correct.

- Using tar pot with wand, install properly heated crack sealer (ASTM 6690) into prepared crack.
- Allow sealer to reach initial set prior to opening area to traffic.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Avoid excessive material that may reduce skid resistance.

A.4 Sealing Concrete Decks

Description

Apply sealant to concrete deck, curbs, sidewalks, and fascia and apply liquid deck sealer where appropriate.

Project Objectives

Maintain waterproof integrity of the deck wearing-surface to prevent water and chlorides from reaching the reinforcing steel.

Labor Skills

Physical labor

Materials

- Water
- Blasting sand
- Sealer (silanes, siloxanes, silicone, polymers, HMWM)
- Steel shot

Equipment

- Paint/herbicide spray unit
- Truck with pumps and spray bar
- Air compressor with hoses, etc.
- Shovels and brooms
- Paint rollers and brushes
- Sandblaster
- Personal safety equipment
- Steel-shot blaster
- Various hand tools

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Loosen and shovel off heavy dirt deposits.
- Clean deck by washing, sweeping, and/or using compressed air.
- Sandblast or shotblast in accordance with sealer manufacturer recommendations.
- Surface must be clean and dry, temperatures in accordance with sealer manufacturer recommendations, and wind calm.
- Apply sealer according to manufacturer's instructions, controlling application rate to avoid running or puddle formation. Use multiple coats, if necessary.

Figure A.4 Sealing Concrete Deck Example (Courtesy of MDOT)

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Use caution because of flammability of some products.
- Vapors of some sealers can be irritants to eyes and throat.

A.5 Replacing Asphalt Wearing Surface

Description

Remove entire existing bituminous wearing surface, and install waterproofing membrane and new bituminous wearing surface.

Project Objectives

Install a wearing surface that provides a smooth skid resistant riding surface and waterproofing protection for underlying structural deck.

Labor Skills

Physical labor

Materials

- Asphalt concrete
- Membrane waterproofing system
- Water
- Blasting sand
- Solvent for cleanup
- Plastic pipe
- Concrete-repair material
- Wire screening/mesh

Equipment

- Dump-truck
- Loader
- Milling machine
- 450-CFM air compressor with hoses, etc.
- Concrete saw
- Electrical generator
- Bituminous spreader
- Roller(s) and plate compactor
- Water tank
- Shovels and rakes
- Core drill
- Squeegees and pails
- Brooms and brushes
- Sandblaster
- Personal safety equipment
- Various hand tools

Procedure

- Check load rating to ensure that bridge can safely support weight of new wearing surface. The load rating should be updated to reflect any additional loads, including additional wearing surfaces.
- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Remove existing wearing surface by milling and/or hand methods.
- Remove material from the site using a loader and heavy dump-trucks.

- Remove dirt and smaller debris by sweeping and shoveling.
- Clean deck with compressed air and prepare surface by sandblasting.
- Make deck repairs as necessary.
- Sandblast repaired areas and again blow the deck clean.
- Drill weep holes and install plastic drain pipes, if necessary.
- Apply waterproof membrane according to manufacturer instructions, depending on type (preformed sheet membrane or liquid membrane).
- Place and roll asphalt concrete (2-inch minimum compacted).

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.

A.6 Lubricating Bearings

Description

Jack structure, clean, and lubricate all appropriate bearings and pin-and-hanger connections. Spot-paint these items if required.

Project Objectives

Ensure that bearings function properly to transfer loads from superstructure to substructure, and allow proper movement of the superstructure.

Labor Skills

Physical labor

Materials

- Water
- Blasting sand
- Paint
- Penetrating oil
- Lubricating oil
- Grease

Equipment

- Hydraulic jacks
- Ladders and scaffolding
- Water trailer
- 150-CFM air compressor with hoses, etc.
- High-pressure water pump
- Sandblaster
- Personal safety equipment
- Stiff brushes
- Various hand tools
- Scaling hammer
- Steel grinder

Procedure

- Develop jacking plan (performed by a licensed engineer).
- Prepare work-zone (i.e., traffic control, environmental protection, ladders and scaffolding, equipment).
- Flush the bearings with high-pressure water or air to remove loose material.
- Remove rust and scale from bearings by scraping, wire brushing, or sandblasting, and re-flush, making sure the bearing is not damaged by rust and scale removal.
- Jack structure pursuant to jacking plan and remove bearings, if necessary for proper service, depending on type of bearing.
- Clean and grind bearing surfaces to a smooth finish.
- Grease or oil bearing wearing surfaces as required, depending on bearing type.

Figure A.5 Penetrating Spray Grease Used on Bearings (Courtesy of PennDOT)

- Prime and paint non-bearing surfaces, as necessary.
- Re-install bearing and lower bridge pursuant to jacking plan, if necessary

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Considerations associated with jacking.
- Restrict traffic during jacking operation.
- Consider environmental and worker safety issues (i.e., asbestos, lead, pathogens).

A.7 Sealing Concrete Substructures

Description

Apply sealant to cap beams, seats, and pedestals, and other substructure elements.

Project Objectives

Maintain waterproof integrity of substructure elements to prevent water and chlorides from penetrating the concrete and reaching the reinforcing steel.

Labor Skills

Physical labor

Materials

- Water
- Blasting sand
- Sealer (silanes, siloxanes, silicone, polymers, epoxies)
- Steel shot

Equipment

- Backpack (handheld) sprayer
- Scrapers and stiff brushes
- Air compressor with hoses, etc.
- Shovels and brooms
- Paint rollers and brushes
- Sandblaster
- Personal safety equipment
- Steel-shot blaster
- Barrel pump with hose and spray attachment
- Various hand tools
- Respirator

Procedure

- Prepare work zone (i.e., traffic control, environmental protection, ladders and scaffolding, equipment).
- Loosen and shovel off heavy dirt deposits.
- Clean substructure elements by washing or using compressed air.
- Sandblast, shotblast, or use stiff brushes and scrapers to loosen material if necessary in accordance with sealer manufacturer recommendations
- The surface must be clean and dry.
- Temperatures within manufacturer requirements and winds calm.
- Apply sealer according to manufacturer instructions. Begin sealing at the bottom of the element and work up, controlling application rate to avoid excess running. Use multiple coats, if necessary.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Use caution because of flammability of some products.
- Vapors of some sealers can be irritants to eyes and throat.

A.8 Painting Bridge Steel

Description

Prepare and repaint paintable bridges.

Figure A.6 Painting Project with Approved Containment System

Project Objectives

Protecting the steel from corrosion and prevent steel section loss.

Labor Skills

- Physical labor
- Industrial painting

Materials

- Approved paint for bridges
- Blasting sand/recycle abrasive (steel shot)

Equipment

- Scrapers, stiff wire brushes, and other hand tools
- Dust containment equipment
- Dust collectors
- Containment system and tarps
- Additional protection equipment for lead paint (wash stations, DECON trailers, etc.)
- Air compressor with hoses, etc.
- Sandblaster
- Abrasives recycle unit
- Sand recovery/Containment
- Paint rollers and brushes
- Personal safety equipment
- Various hand tools
- Power tools (Needle guns, grinders, and scalers)
- Power brushes
- Paint sprayer – airless spray pump
- Ladders/Scaffold

- Wet film thickness gauge

Figure A.7 Wet Film Thickness Gauge

- Prepare work zone (e.g., traffic control, environmental protection, ladders and scaffolding, equipment).

Figure A.8 Containment and Recovery System in Prepared Work Zone

- Remove deteriorated paint using sand or steel shot blasting, power chippers, power brushes, needle scalers, other power tools, or hand tools – vacuum power tools.

Figure A.9 Mechanical Removal of Paint

- Use care to contain dust, blasting sand, and paint chips while preparing steel.

Figure A.10 Interior of the Dust Containment System

- Remove paint and clean to the level specified by the specifications and recommendations for the paint being used.
- The surface and atmospheric conditions must be in accordance with the paint manufacturer recommendations, typically dry surfaces, temperatures over 40 °F, and less than 85 percent humidity.
- Apply paint neatly with roller, brush or sprayer according to manufacturer instructions.

Figure A.11 Example of Painting

- Work paint into all crevices to completely coat steel (stripe coat)
- If multiple coats are necessary, apply following coats after preceding coat is fully cured.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site
- Paint being removed may contain lead, asbestos, or other heavy metals and should be handled appropriately
- Vapors of some paint can be irritants to eyes and throat
- Containment structures, when required, should be constructed in accordance with the project specifications
- Consider environmental and worker-safety issues (e.g., lead exposure)

A.9 Spot Painting Bridge Steel

Description

Prepare and spot paint isolated areas paintable bridges.

Figure A.12 Painting Project with Approved Containment System

Project Objectives

Protecting the steel from corrosion and prevent steel section loss.

Labor Skills

- Physical labor
- Industrial Painting

Materials

- Approved paint for bridges

Equipment

- Scrapers, stiff wire brushes, and other hand tools
- Dust containment equipment
- Dust collectors
- Additional protection equipment for lead paint (wash stations, DECON trailers, etc.)
- Air compressor with hoses, etc.
- Paint rollers and brushes
- Personal safety equipment
- Various hand tools
- Power tools (needle guns, grinders, and scalers)
- Power brushes
- Paint sprayer – airless spray pump
- Ladders/Scaffold
- Wet film thickness gauge

Figure A.13 Wet Film Thickness Gauge

- Prepare work zone (i.e., traffic control, environmental protection, ladders and scaffolding, equipment).
- Remove deteriorated paint using power chippers, power brushes, needle scalers, other power tools, or hand tools – vacuum power tools.

Figure A.14 Mechanical Removal of Paint

- Use care to contain dust, blasting sand, and paint chips while preparing steel.
- Remove paint and clean to the level specified by the specifications and recommendations for the paint being used.
- The surface and atmospheric conditions must be in accordance with the paint manufacturer recommendations, typically dry surfaces, temperatures over 40 °F, and less than 85 percent humidity.
- Apply paint neatly with roller, brush or sprayer according to manufacturer instructions.

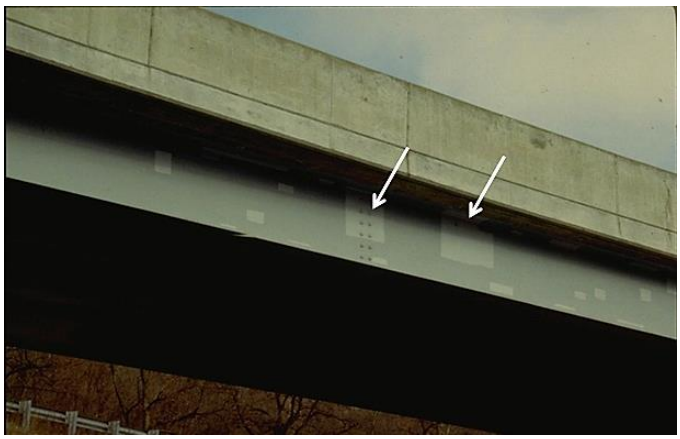

Figure A.15 Example of Spot Painting

- Work paint into all crevices to completely coat steel (stripe coat)
- If multiple coats are necessary, apply following coats after preceding coat is fully cured.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site
- Paint being removed may contain lead, asbestos, or other heavy metals and should be handled appropriately
- Vapors of some paint can be irritants to eyes and throat
- Containment structures, when required, should be constructed in accordance with the project specifications
- Consider environmental and worker-safety issues (e.g., lead exposure)

A.10 Repairing Concrete Decks

Description

Remove and replace damaged portions of structural concrete deck and reinforcing steel.

Project Objectives

To restore structural integrity of the deck, provide a smooth riding surface, and improve safety of the traveling public.

Labor Skills

- Physical labor
- Carpenter
- Cement Masonry
- Ironworker

Materials

- Steel shot
- Blasting sand
- Reinforcing steel
- Water
- Portland cement concrete (or other patching material)

Equipment

- Concrete saw
- 450-CFM air compressor with hoses, etc.
- Electrical generator
- Sandblaster
- Concrete mixer
- Pneumatic hammer (less than 30 pounds)
- Shovels and pickaxe
- Brooms and brushes
- Sounding hammer
- Various hand tools
- Personal-safety equipment
- Steel-shot blaster

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Identify and mark extent of damaged portions. Repair areas should be roughly rectangular.
- Protect area under the deck, if repairs are likely to be full depth prior to demolition
- Sawcut perpendicular to the deck surface outside the damaged area to a depth of 1 inch.
- Remove deteriorated concrete using pneumatic hammers and hand tools to 1 inch below the reinforcing steel.

Figure A.16 Deteriorated Concrete Removal (Courtesy of Kwikbond Polymers)

- Periodically sound the remaining concrete.
- Sandblast or shotblast the concrete substrate to remove loose concrete and roughen surface to provide a sound bond.
- Sandblast exposed steel to remove rust and other contaminants.
- Fasten additional reinforcing steel to the existing steel if section loss is 20 percent or more.
- Form underside of deck for any full-depth repairs.
- Prior to placing patch, the concrete should be saturated surface dry (SSD). If appropriate for the repair material being used, a bonding agent may be applied. Any use of bonding agent must be done in strict conformance with the manufacturer recommendations in order for the bonding agent to perform properly. In particular, the concrete/concrete patch material must be applied within the “open time” of the bonding agent.

Figure A.17 Bonding Agent Application

- Place new concrete or patching material (depending on repair depth) prior to set up of bonding agent.

Figure A.18 Mixing Patching Material (left) and Patch Concrete Placement (right)

- Broom-finish surface of the patch.
- Provide for proper cure to avoid shrinkage cracks.

Figure A.19 Example of Wet Cure

Safety Considerations

Typical work-zone procedures as appropriate for specific site.

A.11 Repairing/Replacing Joints

Description

Repair or remove and replace deteriorated or damaged sections of joint systems, including surrounding concrete. Perform this work on all types of joint systems, as required.

Project Objectives

Provide proper operation of the joint system and safety for the traveling public.

Labor Skills

- Physical labor
- Welding

Materials

- Concrete/elastomeric material
- Blasting sand
- Compression gland, or liquid seal
- Compression-seal lubricant/sealant
- Structural steel shape (armor angle or extrusion)
- Expanded polystyrene board and backer rod
- Epoxy anchor capsules and anchoring devices
- Forming lumber
- Welding rod
- Solvent for cleanup

Equipment

- Welder
- 175-CFM air compressor with hoses, etc.
- Electrical generator
- Sandblaster
- Concrete mixer
- Pneumatic hammer (less than 30 pound)
- Concrete vibrator
- Concrete saw
- Rubber seal installation tools
- Joint levelers
- Hammer drill
- Various hand tools
- Personal-safety equipment
- Oxygen/acetylene or oxygen/propane torches

Procedure

- Prepare work-zone (e.g., traffic control, environmental protection, equipment).
- Sawcut and chip out concrete adjoining joint.

Figure A.20 Chipping out Concrete at Joint

- Remove existing joint system.

Figure A.21 Removed Joint System

- Prepare box out for new joint system.
- Set, level, drill, anchor, and weld armor angle/extrusion.

Figure A.22 Box Preparation for New Joint

- Sandblast all concrete and steel surfaces.
- Place expanded polystyrene board and backer rod (pourable joints).
- Set joint opening at appropriate width. Consult an engineer.
- Place concrete/elastomeric material and cure.
- Install seal.

Safety Considerations

Typical work-zone procedures as appropriate for specific site.

A.12 Repairing/Replacing Steel Members

Description

Repair or replace deteriorated or damaged steel sections.

Project Objectives

To re-establish structural load-carrying capacity of steel bridge elements.

Labor Skills

- Steel Fabrication
- Welding
- Iron Work
- Painting

Materials

- Steel
- Blasting sand
- High-strength bolts
- Welding rod
- Paint

Equipment

- 175-CFM air compressor with hoses, etc.
- Magnetic drill
- Welder
- Paint rollers and brushes
- Scrapers and stiff brushes
- Vacuum paint-removal tools Grinder
- Electrical generator
- Sandblaster
- Oxygen/acetylene or Oxygen/propane torches
- Shroud
- Various hand tools
- Personal-safety equipment
- Jacks may be needed depending on the location and size of the members being repaired or replaced. Jacking may be needed to remove load from member so it can be repaired/replaced.
- Cranes may be needed depending on the location and size of the members being repaired. Large pieces of steel will require a crane.

Procedure

- Develop repair plan.
- CONSULT A LICENSED PROFESSIONAL ENGINEER.
- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Provide temporary support for affected members, as necessary. Figure A.19 shows temporary support of truss end post

Figure A.23 Temporary Support

- Prepare steel area to be repaired by removing paint, rust, dirt, etc.
- Remove paint using vacuum-shrouded power tools.
- Remove rust using vacuum-shrouded tools, wire brushes, scrapers.
- Remove loose dirt and grime with compressed air or high-pressure water.
- Perform repair as required.

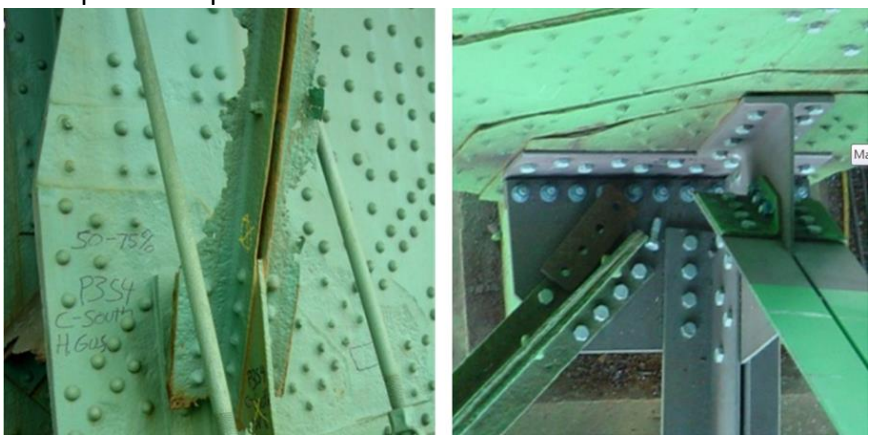

Figure A.24 Example of Steel Member Repair

- CAUTION: Only certified welders should weld structural steel members.
- Conduct any necessary non-destructive testing.
- Prime and spot-paint treated area.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- DO NOT attempt steel repairs without proper knowledge and qualifications.
- Consider environmental and worker-safety issues (i.e., lead exposure).

A.13 Repairing Sheared Anchor Bolts/Replacing Bearings

Description

Jack structure and repair or replace bearing systems or system components for all types of bearings, as required.

Project Objectives

Ensure that bearings function properly to transfer loads from superstructure to substructure, and allow proper movement of the superstructure.

Labor Skills

- Physical labor
- Iron worker

Materials

- Steel flat stock
- Threaded rod
- Chemical Grout
- Anchor bolts
- Grease
- Paint
- Bolts/nuts
- Concrete (or other patching material)
- Water
- New elastomeric bearings or other replacement bearings

Equipment

- Hydraulic jacks
- Grinders
- Magnetic drill
- Air compressor with hoses, etc.
- Electrical generator
- Scalers, wire brushes
- Ladders and scaffolding
- Various hand tools
- Oxygen/acetylene torches
- Personal safety equipment
- Welder mask, gloves, apron
- High-pressure water pump with hoses, etc.

Procedure

1. Repair sheared anchor bolts on slider bearings by installing anchor straps in front of bearing base plates and properly lubricating sliding surfaces.
   - Develop repair plan, including jacking plan. Consult a licensed engineer.
   - Measure pedestal width and hole locations.
   - In shop, cut straps to size, drill holes in straps, and paint.
   - Prepare work-zone (i.e., traffic control, environmental protection, ladders and scaffolding, equipment).

- Drill holes in pedestal at planned strap locations to required depth.
- Set threaded rods in holes using chemical anchor. Be sure rods are properly aligned.
- Jack structure pursuant to jacking plan and remove bearings.

Figure A.25 Example of Jacked Structure

- Remove rust and scale from bearings by scraping and wire brushing, and re-flush, making sure the bearing is not damaged by rust and scale removal.
- Reinstall bearings and install anchor straps.
- Place dry lube pads and grease on sliding surface.
- Lower structure pursuant to jacking plan.
- Prime and paint bearing non-wearing surfaces as necessary.

2. Replace roller-nest bearings with elastomeric bearings or pads.

- Develop repair plan, including jacking plan. Consult a licensed engineer.
- Fabricate filler plates for new bearings in shop, based on repair plan.
- Prepare work-zone (i.e., traffic control, environmental protection, ladders and scaffolding, equipment).
- Jack structure pursuant to jacking plan.

Figure A.26 Example of Jacked Structure for Bearing Replacement

- Remove old bearings.

Figure A.27 Example of Bearing Removal

- Drill holes in pedestal and install new anchor bolts.
- Install new elastomeric pads or elastomeric bearings and filler plates, as necessary.
- Lower structure pursuant to jacking plan.
- Prime and paint bearing non-wearing surfaces, as necessary.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Restrict traffic during jacking operation.
- Consider environmental and worker-safety issues (e.g., asbestos exposure).

A.14 Repairing Concrete Substructures

Description

Remove and replace damaged portions of substructure concrete and reinforcing steel.

Project Objectives

To restore structural integrity of the substructure and bridge.

Labor Skills

- Physical labor
- Carpenter
- Cement masonry
- Ironworker

Materials

- Blasting sand
- Shim plate
- Reinforcing steel
- Water
- Portland cement concrete (or other patching material)

Equipment

- Concrete saw
- 450-CFM air compressor with hoses, etc.
- Electrical generator
- Sandblaster
- Sounding hammer
- Pneumatic hammer
- Concrete mixer
- Concrete pump
- Hydraulic pump
- Brooms and brushes
- Various hand tools
- Personal-safety equipment

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Identify and mark extent of damaged areas of substructure concrete.
- Install temporary support of superstructure as required.
- Sawcut outside (perimeter) deteriorated areas. Cuts should be dovetailed to hold repair concrete.
- Remove deteriorated concrete, using pneumatic hammers and hand tools, and expose reinforcing steel to allow placing of concrete behind reinforcing steel - 1 inch beyond steel.

Figure A.28 Deteriorated Concrete Removal

- Periodically sound remaining concrete to determine concrete condition.
- Clean the area using sandblasting.
- Fasten additional reinforcing steel to replace or repair existing steel as necessary to reestablish steel to as-built condition.
- Form area to be repaired.

Figure A.29 Examples of Forms

- Saturate the concrete substrate to a Saturated Surface Dry (SSD) condition.
- Apply grout or other bonding agent to existing concrete surfaces if recommended for use with patch material and consistent with local practice. If bonding agent is used it must be applied in strict compliance with manufacturer recommendations.
- Place new concrete or patching material. (Apply pneumatic concrete as an alternative).
- Provide proper cure to ensure durable concrete and to avoid shrinkage cracking.
- Remove forms.
- Patch any surface defects resulting from forming.

Safety Considerations

Typical work-zone procedures as appropriate for specific site.

A.15 Repairing Erosion or Scour

Description

Repair undermined foundations and/or scoured or eroded stream channels with concrete, grout, stone fill, or riprap.

Project Objectives

To protect integrity of bridge substructures and to ensure they continue to function as intended.

Labor Skills

Physical labor

Materials

- Gravel
- Bedding material
- Geotextile fabric
- Rip-rap
- Concrete materials
- Heavy stone fill
- Grout
- Grout bags
- Bulldozer
- Skid-steer loader
- Backhoe
- Hydraulic excavator
- Dump truck(s)
- Crane
- Barge
- Concrete pump
- Core drill
- Various hand tools
- Grout mixer
- Personal-safety equipment

Procedure

1. **Repair Undermined Foundation**

   - Coordinate with Agency Environmental Unit and obtain any necessary permits from State Environmental Department, and/or US Army Corps of Engineers, and consult with environmental agencies, as required.
   - Also coordinate with the Agency Geotechnical Engineering, structural engineering and Hydraulic Engineering staff on a repair procedure.
   - Prepare work-zone (i.e., traffic control, environmental protection, temporary coffer dams, equipment).
   - Construct cofferdams and de-water unless using tremie concrete or grout bags
   - Remove silt and other fine material deposited under the foundation.

- Place sand bags, forms, or bagged concrete along vertical face of foundation.
- Drill holes through footing, approximately 3-feet apart.
- Pump concrete or grout through holes, vibrating concrete to consolidate it.
- Remove forms.
- As alternative, place grout bags along vertical face of foundation and pump concrete into grout bags until full. Repeat as necessary.
- Place stone fill or riprap protection around foundation as noted below.

2. Repair Scour Hole

- Coordinate with Agency Environmental Unit and obtain any necessary permits from State Environmental Department, and/or US Army Corps of Engineers, and consult with environmental agencies, as required.
- Also coordinate with the Agency Geotechnical Engineering and or Hydraulic Engineering staff on a repair procedure.
- Prepare work-zone (i.e., traffic control, environmental protection, temporary coffer dams, equipment).
- Remove silt and other fine material deposited into the scour hole, if possible.
- Place gravel or bedding material in the scour hole, as required. Place geotextile fabric over the bedding material.
- Place stone into scour hole. Shape stone as close as possible to existing channel elevation. Consult an engineer to determine the appropriate size riprap for stream conditions.
- Safety Considerations: Typical work-zone procedures as appropriate for specific site
- Work near water requires additional precautions such as Personal Floatation Devices (PFD's), ring buoys, skiffs, fall protection, etc.

A.16 Placing Thin Polymer Overlays

Description

Apply thin polymer overlay to concrete deck where appropriate.

Project Objectives

Maintain waterproof integrity of the deck wearing-surface portion to prevent water and chlorides from reaching the reinforcing steel.

Labor Skills

Physical labor

Materials

- Polymer Components
- Blasting sand
- Aggregates
- Steel shot
- Solvent for cleanup
- Sand

Equipment

- Air compressor with hoses, etc.
- Mixing paddle
- Sandblaster
- Screed
- Steel-shot blaster
- Spiked shoes
- Shovels, scrapers, and brooms
- Personal safety equipment
- Rakes, squeegees, and pails
- Respiratory equipment
- Mortar mixer or bucket
- Various hand tools

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Loosen and shovel off heavy dirt deposits.
- Clean deck by washing, sweeping, and/or using compressed air.
- Steel shot blast and sandblast.
- Apply test patches to verify cleaning method.
- Surface must be clean, dry, and temperatures must be within polymer manufacturer specifications.
- Apply polymer according to manufacturer instructions, controlling application rate to avoid running and irregular surfaces.
- Apply second coat if necessary.
- Broadcast aggregate on surface of overlay to create a friction surface.

Figure A.30 Broadcasting Aggregate by Hand

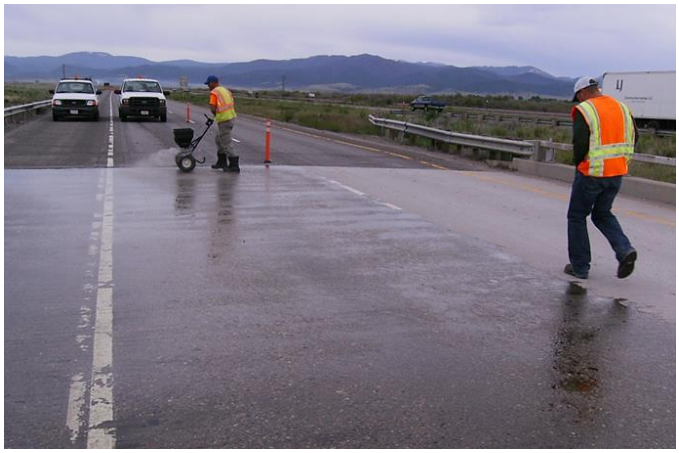

Figure A.31 Broadcasting Aggregate with Hand Operated Spreader (Courtesy of Kwikbond Polymers)

Figure A.32 Broadcasting Sand using a Mechanized Application (Courtesy of MDOT)

- Allow sufficient time for overlay to cure before permitting traffic on it.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Use caution because of flammability of some products.

A.17 Shotcrete Repairs

Description

Application of pneumatically placed Portland cement/sand mixture for structural concrete repairs.

Project Objectives

- Provide form free structural repairs to concrete structures.

Labor Skills

- Physical labor
- Skill/training of operator

Materials

- Type I or II Portland cement
- Potable water
- Clean/washed mason sand
- Waterproofing cementing compound
- Curing agent

Equipment

- Air compressor
- Coalescing filter
- Sandblaster
- Various hand tools
- Pneumatic hammer (less than 30 pounds)
- Brooms and brushes
- Sounding hammer
- Shotcrete
- Application equipment
- Personal-safety equipment

Figure A.33 Shotcrete Mixing Equipment

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Identify and mark extent of damaged portions.
- Test patches to verify ability of operator.
- Remove deteriorated concrete using pneumatic hammers and hand tools to 1 inch below the reinforcing steel.
- Periodically sound the remaining concrete.
- Clean the area using sandblasting.
- Sandblast exposed steel to remove rust and other contaminants.
- Fasten additional reinforcing steel to the existing steel if section loss is 20 percent or more.
- Existing concrete should be Saturated Surface Dry (SSD) prior to application of neat cement paste, or sand cement grout mix. Follow manufacturer recommendations for waterproof cementing compound, or epoxy bonding agent.
- Screen/mix the Portland cement and mason sand.
- Apply mix to chipped area with low volume shotcrete applicator gun.

Figure A.34 Shotcrete Applicator Gun

- Screed/finish the new repair
- Apply Curing agent coat to repaired area, unless wet cure is being used. If wet cure is being used, cover appropriately and keep wet for curing period
- Daily clean low volume shotcrete unit at the end of the day

Safety Considerations

- Typical work-zone procedures as appropriate for specific site
- Fall safety procedures
- Scaffolding erection procedures

A.18 Full Depth Bridge Deck Repairs

Description

Perform full depth repairs of bridge decks as necessary.

Project Objectives

Extend service life of existing structure until future major rehabilitation. Improve rideability of wearing surface.

Labor Skills

- Physical labor
- Carpenter
- Cement mason
- Ironworker

Materials

- Blasting sand
- Reinforcing steel
- Water
- Portland cement concrete (or other patching material)

Equipment

- Concrete saw
- 450-CFM air compressor with hoses, etc.
- Electrical generator
- Sandblaster
- Concrete mixer
- Pneumatic hammer (less than 30 pounds)
- Shovels and pickaxes
- Brooms and brushes
- Sounding hammer
- Various hand tools
- Personal-safety equipment

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Identify and mark extent of damaged portions. Repair areas should be roughly rectangular.
- Sawcut outside the damaged area (straight cuts) – 1 inch.
- Remove deteriorated concrete using pneumatic hammers.
- Periodically sound the remaining concrete.
- Clean the area using sandblasting.
- Sandblast exposed steel to remove rust and other contaminants.

Figure A.35 Deteriorated Concrete Removal and Exposed Steel Cleaned

- Fasten additional reinforcing steel to the existing steel if section loss is 20 percent or more.
- Construct form on underside of deck repairs. Ensure there is sufficient bracing on bottom side of deck form to avoid failure of formwork when concrete is placed. A heavy gauge tie wire (9 wire) can be used to help tie form to existing rebar.

Figure A.36 Example of Form Constructed on Deck Underside

- Make sure repair area is clean.
- Follow repair material manufacturer recommendation for surface preparation (wet hole, dry hole).
- Available repair materials include Portland cement concrete, Elastomeric concrete, or "Liquid" concrete.
- Mix repair material as per manufacturer recommendations placed in full depth repair.
- Fill repair, consolidate as recommended, then finish surface.
- Place aggregate for friction on surface as necessary.
- Open deck back up to traffic following appropriate cure time (again as per manufacturer recommendation) and strength gain.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site
- Fall safety procedures
- Scaffolding erection procedures

A.19 Bonded FRP Repairs to Concrete

Description

Repair damaged portions of concrete.

Project Objectives

To restore structural integrity of the concrete.

Labor Skills

- Physical labor
- Carpenter
- Mason

Materials

- Glass or carbon fiber fabric
- Polymer resin and hardener
- Solvent for cleanup
- Blasting sand
- Water

Equipment

- Pressure washer
- Sandblaster
- Sounding hammer
- Pneumatic hammer
- Hand roller
- Various hand tools
- Personal-safety equipment
- Resin manufacturer recommended equipment for application and impregnation of resin
- Respirators

Procedure

- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Identify and mark extent of damaged areas of concrete.
- Surface preparation and column wrap should include the full exposed column height plus 2 feet below ground.
- All concrete surfaces should be repaired (including spalls and delaminations) and epoxy injection crack sealing performed. Surface should be free of sharp edges that can damage the fabric. Surface voids and depressions should be filled with epoxy.
- Surface should be completely dry at the time of application of fabric. Newly repaired or patched surface should be cured at least 7 days prior to wrapping.
- One prime coat of manufacturer resin should be applied to the surface and should be allowed to become tacky to touch.
- Fabric, usually about 2 to 3 feet wide, should be saturated at the job site with resin (usually epoxy) using the specified fiber-resin ratio.

- Saturated fabric should be wrapped around the column by hand lay-up, using methods that produce a uniform, constant tensile force that is distributed across the entire width of fabric.

Figure A.37 Hand Lay-Up of Saturated Fabric

- Entrapped air, if any, should be rolled out before the resin sets.
- Subsequent layer(s) should be applied continuously (or spliced) until specified number of layers is achieved at a section. Adjacent sections should use a butt joint.
- The system should be protected against water and rainfall for at least 4 days following installation.
- After the system is cured, a protective topcoat is applied to the fiberwrap surface.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site
- Fall safety procedures
- Scaffolding erection procedures
- Vapors of some resins can be irritants to eyes and throat

A.20 Repairing Paved Slope Protection

Description

Repair undermined or eroded paved slope protection with concrete, grout, precast concrete blocks or stone slope paving.

Project Objectives

To protect integrity of bridge substructures by reducing the probability that soils will be erode from beneath and around the abutment.

Labor Skills

Physical labor

Materials

- Gravel
- Geotextile Fabric
- Rip-rap
- Concrete
- Precast Concrete Pavers
- Grout
- Air Compressor
- Chipping hammer
- Concrete saw
- Backhoe
- Plate Tamper
- Dump truck(s)
- Concrete pump
- Core drill
- Various hand tools
- Grout mixer
- Personal-safety equipment

Procedure

- Coordinate with Agency Environmental Unit and obtain any necessary permits from State Environmental Department, and/or US Army Corps of Engineers, and consult with environmental agencies, as required.
- Coordinate with the Agency Geotechnical Engineering and or Hydraulic Engineering staff on a repair procedure.
- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Remove damaged and unstable slope paving.
- Sound existing slope paving to determine if there is any undermining, or voids. Depending on the extent of the voids the choice must be made whether to try to fill the void with the slope protection in place, or remove the slope protection.
- For slope protection with voids that is remaining in place, Drill holes through paving approximately, approximately 3-feet apart.
- Pump concrete or grout through holes, vibrating concrete to consolidate it.

- In areas where slope paving has been removed, the soil beneath should be filled with gravel or crushed stone and compacted
- Place the new slope paving on the compacted stone/gravel taking care to match the grades of the existing slope paving. Form, if the repair is cast in place concrete, place pavers if they are being used.
- Remove forms as required.
- Safety Considerations: Typical work-zone procedures as appropriate for specific site
- Work near water requires additional precautions such as Personal Floatation Devices (PFD's), ring buoys, skiffs, fall protection, etc.

Figure A.38 Paved Abutment Slope Repair. Before repair (left) and After (right). (Courtesy of Georgia DOT)

A.21 Repairing Stone Slope Protection

Description

Repair undermined or eroded stone slope protection with new riprap (stone fill).

Project Objectives

To protect integrity of bridge substructures by reducing the probability that soils erode beneath and around the abutment.

Labor Skills

Physical labor

Materials

- Gravel/crushed stone
- Geotextile Fabric
- Rip-rap
- Grout
- Backhoe
- Hydraulic Excavator
- Plate Tamper
- Dump truck(s)
- Concrete pump
- Core drill
- Various hand tools
- Grout mixer
- Personal-safety equipment

Procedure

- Coordinate with Agency Environmental Unit and obtain any necessary permits from State Environmental Department, and/or US Army Corps of Engineers, and consult with environmental agencies, as required.
- Coordinate with the Agency Geotechnical Engineering and or Hydraulic Engineering staff on a repair procedure.
- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Remove loose debris, soil or vegetation in the area to be repaired.
- Fill any gullies, voids, or erosion holes with compacted gravel/crushed stone.
- If it is the agency practice to use a bedding geotextile, place it over the gravel.
- Place riprap to the lines and grades required to match existing riprap slope protection
- If riprap slope protection is grouted, place grout in voids between pieces of riprap stone.
- Safety Considerations: Typical work-zone procedures as appropriate for specific site
- Work near water requires additional precautions such as Personal Floatation Devices (PFD's), ring buoys, skiffs, fall protection, etc.

Figure A.39 Stone Abutment Slope Repair. Before Repair (left) and After (right). (Courtesy of Georgia DOT)

A.22 Repairing/Replacing Steel Truss Ends

Description

Repair or replace deteriorated or damaged steel at ends of truss.

Project Objectives

To re-establish structural load-carrying capacity of steel truss.

Labor Skills

- Steel fabrication
- Welding
- Iron work
- Painting

Materials

- Steel
- Blasting sand
- High-strength bolts
- Welding rod
- Paint

Equipment

- 175-CFM air compressor with hoses, etc.
- Magnetic drill
- Welder
- Paint rollers and brushes
- Scrapers and stiff brushes
- Vacuum paint-removal tools, grinder
- Electrical generator
- Sandblaster
- Oxy-acetylene torch/propane torches
- Shroud
- Cut off saw
- Various hand tools
- Personal-safety equipment

Procedure

- Develop repair plan.
- CONSULT A LICENSED PROFESSIONAL ENGINEER.
- Prepare work-zone (i.e., traffic control, environmental protection, equipment).
- Provide temporary support for affected members, as necessary.
- Carefully remove any damaged or deteriorated members that are to be replaced in a manner that will not damage any adjacent members to be retained or repaired. Removal may require flame cutting, drilling, or sawing.
- Prepare steel area to be repaired by removing paint, rust, dirt, etc.
- Remove paint using vacuum-shrouded power tools.
- Remove rust using vacuum-shrouded tools, wire brushes, scrapers.
- Remove loose dirt and grime with compressed air or high-pressure water.

- Replace or repair pins, gusset plates, eye-bars, tension members, compression members, as called for in the repair plan.

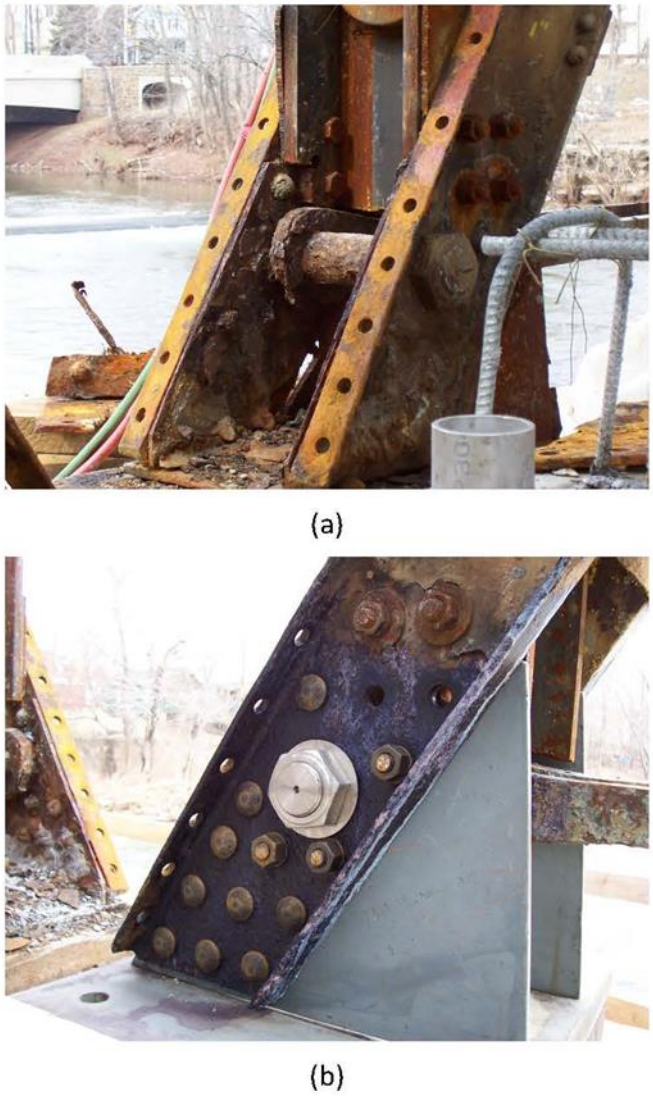

Figure A.40 Example of Truss End Repair (a) Before and (b) After

- CAUTION: only certified welders should weld structural steel members.
- Conduct any necessary non-destructive testing.
- Prime and spot-paint treated area.

Safety Considerations

- Typical work-zone procedures as appropriate for specific site.
- Truss End Repairs will require an engineer to develop a repair plan as well as a plan to support the truss during repairs.
- DO NOT attempt steel repairs without proper knowledge and qualifications.
- Consider environmental and worker-safety issues (i.e., lead exposure).

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Appendix B -References

B.1 Example Inspection Report

The following report is an example of what a bridge inspection reportcould look like. State DOT's have their own formatting for these documents. This report is a generic example. This example is for illustration purposes and may not reflect current requirements of the National Bridge Inventory Standards or AASHTO bridge inspection related manuals.

Example Inspection Report

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This example report is a compilation of examples from seven different state departments of transportation. It represents several bridges, whereas a typical bridge inspection report would represent just a single bridge.

Page 1

General Description of Bridge

Your-State Department of Transportation

Bridge Inspection Report

District	07	**Structure**	Sixes River, Hwy 9	**Bridge ID**	00901 C
Fac Crossed	Sixes River	**Owner**	State Highway Agency	**Fac Carried**	US101 (HWY009)
Suff Rating	56.9	**County**	Durry	**Mile Point**	295.75 mi.
AC Depth	0.00	**Record Type**	1	**Insp Date**	05/20/2013
Bridge Length	500.00 ft.	**Insp Freq**	24	**Inspector 1**	Name 1
		Bridge Width	50.50 ft.	**Inspector 2**	Name 2

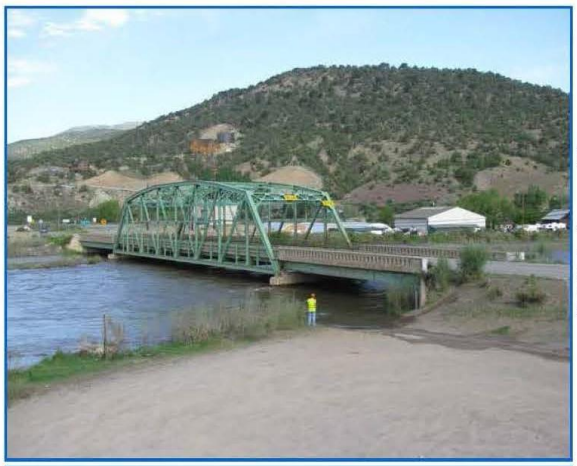
Signature:

Typical information contained in the "General Description" portion of the report

Page 2

General Photos

Typically orientation of photo given



F08F-b NW.jpg

Taken: 2008/06/03 5:50
Posted: 2009/02/13
File Size: 1067938
e:\web\folders\F-08-F\

F08F-a West.jpg

Taken: 2008/06/03 5:53
Posted: 2009/02/13
File Size: 1167298
e:\web\folders\F-08-F\

Important to know when the photo was taken and where is it located

Typical inspection reports contain general photos that show overall layout.

Page 3

Element Condition States Example 1

BMS Elements

Element	Element Description	Total	Units	State 1	State 2	State 3	State 4
13	Bridge Deck Surface	3,562	SF	3,562	0	0	0
38	Concrete Slab	3,562	SF	3,562	0	0	0
206	Timber Pile/Column	8	EA	7	0	1	0
215	Concrete Abutment	60	LF	60	0	0	0
228	Timber Submerged Pile/Column	24	EA	20	1	2	1
234	Concrete Pier Cap/Crossbeam	87	LF	87	0	0	0
322	Bridge Impact	2	EA	0	1	1	0
330	Metal Bridge Railing	274	LF	274	0	0	0
361	Scour	3	EA	0	3	0	0
400	Asphalt Butt Joint Seal	52	LF	0	0	52	0

These element condition states quantify the extent of deterioration found **by the inspectors**

Total quantities and how they are measured. LF is linear foot, EA is each, SF is square foot.

The total of all condition **state quantities** add up to the element total quantity.

Page 4

Element Condition States Example 2

There are four condition states (CS)

Element Condition States (New AASHTO report)

Elemkey	Defects/Prot Sys	Quantity	Units	Env	CS 1	CS 2	CS 3	CS 4
12-RC Deck		**25250**	**(SF)**	**Sev.**	**99%**	**1%**	**0%**	**0%**
	1080-Spalls/Delams/Patched Areas	1	(SF)	Sev.	99%	1%	0%	0%
	1130-Cracking (RC and Other)	1	(SF)	Sev.	100%	0%	0%	0%
109-Prestressed Girder / Beam		**3500**	**(LF)**	**Sev.**	**99%**	**0%**	**1%**	**0%**
	1080-Spalls/Delams/Patched Areas	1	(LF)	Sev.	98%	1%	1%	0%
	1090-Exposed Rebar	1	(LF)	Sev.	99%	0%	1%	0%
	1110-Cracking (PSC)	1	(LF)	Sev.	99%	0%	1%	0%
210-RC Pier Wall		**165**	**(LF)**	**Sev.**	**80%**	**20%**	**0%**	**0%**
	1130-Cracking (RC and Other)	1	(LF)	Sev.	90%	10%	0%	0%
	1180-Abrasion	1	(LF)	Sev.	90%	10%	0%	0%
215-RC Abutment		**110**	**(LF)**	**Sev.**	**100%**	**0%**	**0%**	**0%**
302-Compressn Joint Seal		**101**	**(LF)**	**Sev.**	**0%**	**100%**	**0%**	**0%**
	2310-Leakage	1	(LF)	Sev.	0%	100%	0%	0%
	2350-Debris Impaction	1	(LF)	Sev.	0%	100%	0%	0%
	2370-Metal Damage	1	(LF)	Sev.	0%	100%	0%	0%
306-Other Joint		**100**	**(LF)**	**Sev.**	**0%**	**0%**	**0%**	**100%**
	2310-Leakage	1	(LF)	Sev.	0%	0%	0%	100%
310-Elastomeric Bearing		**56**	**(EA)**	**Sev.**	**100%**	**0%**	**0%**	**0%**

Elements and defects are given numerical assignments that help with data sorting

In this report, specific types of defects have an identification number that can be selected from a dropdown box

Page 5

Inspection Findings

Highway Number (ON) 5D: 070A 1

Mile Post (ON)11: 62.887 mi

Department of Transportation

Structure Inspection and Inventory Report (English Units)

Elem/Env	Description	Element Notes
113/1	Paint Stl Stringer	In relatively good condition. Speckled rust at ends of some exteriors, and at a few interiors. The top of the stringers are coped around floor-beam flanges, though no problems. BROKEN Rivets at floor beam connections: Stringer 3b has 3 heads broken off at FB2 & 1 at FB3.
121/1	P/Stl Thru Truss/Bot	Spots of light surface rust and freckling rust full length, esppecia there are several pigeon nests on the lower batten plates, 10-20 The drain tubes extend below these lower chords, good. There are trees through & against the panel 1 lower chords, prol dirt on them, creates shade. From the Special 2008 Gusset Plate inspection :-) The L6 Right, Span 1 interior gusset plate has a 4 inch X 16 inch area of R3 corrosion, with 15% to 20% section loss along the connection to the top flange of the lower chord. The L5 Right, Span 2 gusset plate has a 3 inch diameter area of R2 to R3 corrosion, with 10% to 15% section loss on the interior gus diagonal L5 to U4.
126/1	P/Stl Thru Truss/Top	Some light surface rust in spots, with freckling rust full leng 1/2, especially surfaces exposed to plow spray; (see 10-20 diagonals). Minor damage: East portal, Span 2 and sway brace 3 in Sp high loads.
152/1	Paint Stl Floor Beam	Light and moderate rust primarily in the upper flanges (worst is L3, Span 1), and ends of lower flanges of most. R2 corr. at the ends of some floor beams, 2010 PHOTO, but they are relatively free of dirt. Thin cover plates welded to bottom flanges.
210/1	R/Conc Pier Wall	Two 8 foot walls, with a debris wall between them. Light shrinkage cracking on both walls. The debris wall has a few hairline and vertical cracks on the Rear side, and a 2 foot X 1 foot X 1 inch deep spall with exposed rebar, on the Forward face.
215/1	R/Conc Abutment	Both are water stained with several light vertical cracks. Light stream abrasion on the bottom of A3.
304/1	Open Expansion Joint	A sliding steel plate above P2 has patches & some small potholes along it, see 2010 PHOTO. The remainder of the length has cracked asphalt along it. The asphalt cover is open 1 to 3.5 inches along the actual joint. It was never intended to seal.

Inspection notes don't always have to state a deficiency

Findings are usually quantified by measurements or percentages

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Highway Number (ON) 5D: 070A 1

Department of Transportation

Mile Post (ON)11: 62.887 mi

Structure Inspection and Inventory Report (English Units)

Elem/Env	Description	Element Notes
308/1	Constr Non Exp Jt	Construction joints above each floor beam, except Pier 2. Covered with asphalt except at edges near curbs. Sealant along the majority of asphalt cracks, previous evidence of leaking on corroded floor beams, not visibly open or D-cracked, good. Abutment joints are cracked, patched, sealant applied and depressed, but impending potholes again.
311/1	Moveable Bearing	At the Pier, rockers are tipped in the expansion position. R1 corr. on all, relatively clean; (see 10/2006 PHOTO).
313/1	Fixed Bearing	At the abutments - Spotted R1 corrosion. Look OK, relatively clean but R2 corr. on the anchor bolt nut, on Bearing 3B, with R1 corr. on the plate.
32(		All OK.
33(		Faded painted rail is peeling throughout, with some freckling and flaking. Pipe top rail with angle & channel rail, not the typical lattice rail.
338/1	Conc Curbs/SW	Heavy scale, primarily in Span 1, Right side, panel 3 being the worst (see 10-2006 PHOTO), also Panel 6 of Span 1. Moderate scale in many spots elsewhere. A few transverse cracks. Both ends at A3 are spalled with exposed rebar and there is a broken end at the Pier 2 joint, left curb.
359/1	Soffit Smart Flag	A few short transverse cracks. Areas of light to moderate map cracking and scale, with light to moderate efflor.; (see 10-2006 PHOTO). The haunch concrete is popped off at a few stringer to floor beam connections.
501/1	Channel Cond	Colorado River, fair to poor alignment; large river with cobble bed has cut to the East and is now adjacent to Abutment 3; (see 10-2006 PHOTOS). The river meanders easily in this region. There are cobble bars with maturing vegetation. The primary flow is in Span 2 during the 2008 & 2010 inspections.
502/1	ChannProtMatCond	Some placed concrete and small rounded lava rock riprap, just upstream of Abutment 3 is inadequate for the strong flows.
504/1	BankCond	Lined with trees and bushes.
505/1	Debris Smart Flag	Some logs on the banks, and downstream side of Span 1 now.

Photos showing defects are very helpful and are often referenced in the defect description.

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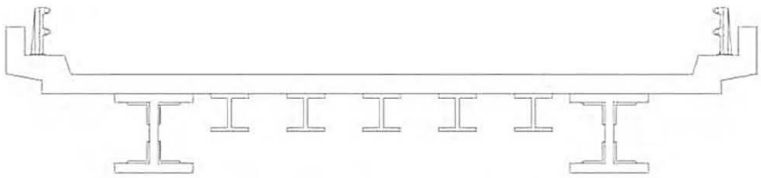
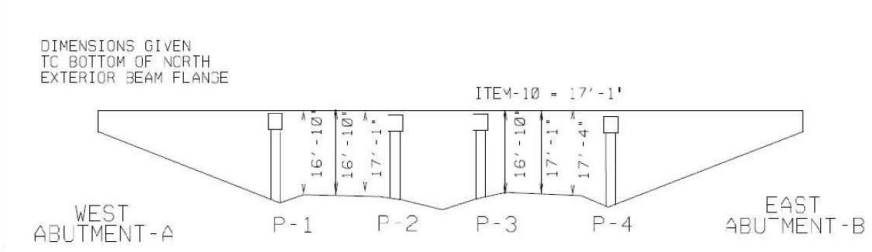
Bridge Inspection Field Sketches





Simple sketches help to clarify locations of defects, define elements, or help with measurements such as clearances

Condition Photos

STRUCTURE INSPECTION REPORT
PHOTOS

Agency ID: **Date of Inspection:** March 28, 2012

Photo-1
Shows delamination of the epoxy overlay/repair typical at Pier-3 and 4.

Photo-2
Shows delamination and shallow spalling to the deck in Span-2.

Showing a tool, or other item of known size, provides a reference to the scale shown in the photo.

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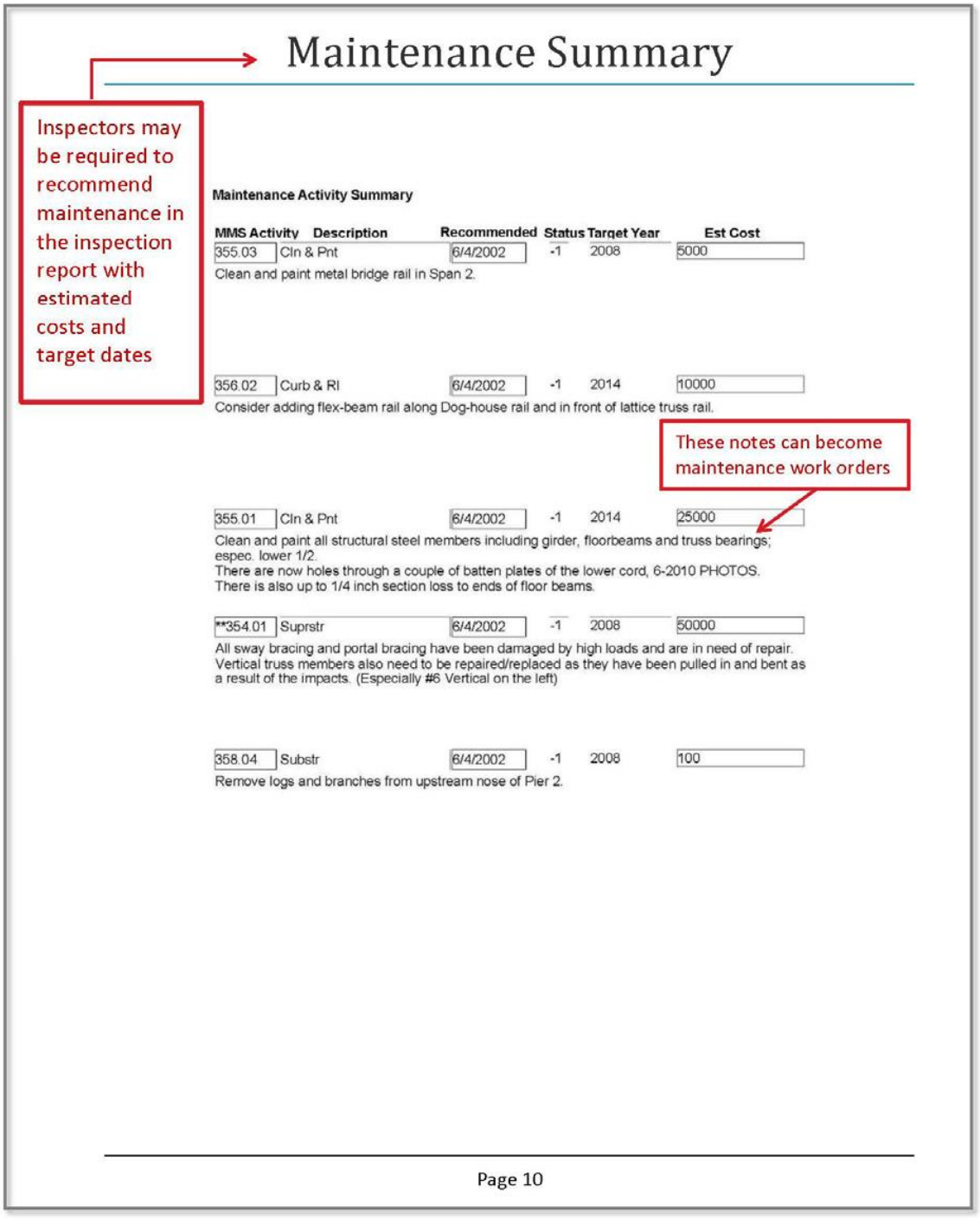

Maintenance Summary

Inspectors may be required to recommend maintenance in the inspection report with estimated costs and target dates

Maintenance Activity Summary

MMS Activity	Description	Recommended	Status	Target Year	Est Cost
355.03	Cln & Pnt	6/4/2002	-1	2008	5000

Clean and paint metal bridge rail in Span 2.

356.02	Curb & Rl	6/4/2002	-1	2014	10000

Consider adding flex-beam rail along Dog-house rail and in front of lattice truss rail.

These notes can become maintenance work orders

355.01	Cln & Pnt	6/4/2002	-1	2014	25000

Clean and paint all structural steel members including girder, floorbeams and truss bearings; espec. lower 1/2.
There are now holes through a couple of batten plates of the lower cord, 6-2010 PHOTOS.
There is also up to 1/4 inch section loss to ends of floor beams.

**354.01	Suprstr	6/4/2002	-1	2008	50000

All sway bracing and portal bracing have been damaged by high loads and are in need of repair. Vertical truss members also need to be repaired/replaced as they have been pulled in and bent as a result of the impacts. (Especially #6 Vertical on the left)

358.04	Substr	6/4/2002	-1	2008	100

Remove logs and branches from upstream nose of Pier 2.

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Crew Recommendations

WORK RECOMMENDATIONS

CREW RECOMMENDATIONS			CONTRACT RECOMMENDATIONS		
Deck Patching			Bridge Replacement		
Approach Pavement			Superstructure Replacement		
Joint Repair	M	Seal joints.	Deck Replacement		
Railing Repair			Overlay		
Detailed Insp			Widen		
Zone Paint			Paint		
Substr. Repair			Zone Paint		
Slope Repair			Pin and Hanger		
Brush Cut			Substructure Repair		
Other Crew Work			Other Contract Work	M	Replace north thrie beam at impact zones.

This example from an inspection report separates in-house maintenance crew work from contracted work. For this bridge, it is recommended that maintenance forces seal the joints while a contractor replace a portion of the bridge rail.

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Load Rating Summary

Calculations are either performed or reviewed as part of the inspection to make sure the bridge can safely pass all loads or may need posting

Load Rating Summary Form

Rte.: _____
Over: _____
County: County
Str. No.: _____
Method: Load Factor
Rated by: . Date: 07/03/1996
Checked by: _____ Date: _____

POSTING RATING – State X Legal Loads
(at 75 % Yield)

Single Unit 52.1 Tons Controlling Member G06 @ S01

Truck and Semi-trailer 77.4 Tons Controlling Member G06 @ S01

NBIS RATINGS
Note: The rating is the gross tonnage on a HS20 vehicle.

HS20 at Inventory 30.47 Tons Controlling Member G06 @ S01

HS20 at Operating 50.79 Tons Controlling Member G06 @ S01

BLANKET PERMIT RATINGS
(at Operating Level)

90,000 lb vehicle 69.1 Tons Controlling Member G06 @ S01

115,000 lb vehicle 80.7 Tons Controlling Member G06 @ S02

Comments:

Signature: _____
Load Rater

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B.2 Summary of Condition States and Feasible Actions

Bridge condition is reported using a single scale that reflects common process for deterioration and the effect on serviceability. The general pattern for a Bridge Element having four condition states is as follows:

1. Good – No deterioration to minor deterioration
2. Fair – Minor to Moderate deterioration
3. Poor – Moderate to Severe deterioration
4. Severe – Beyond the limits established in condition state 3 and/or warrants a structural review to determine strength or serviceability of the element or bridge

Each of these levels of deterioration is called a condition state. The condition state methodology provides two steps of information about a bridge element's deterioration:

- Severity – characterized by precise definition of each condition state
- Extent – the distribution of the total element quantity among condition

The severity is important for selection of a feasible and cost effective preservation treatment, and extent is important for cost estimation.

Assignment of quantities to condition states is determined from element definitions and element commentary for National Bridge Elements. Condition state definitions are guidelines to the bridge inspector for categorization of the severity of the deficiency. Element commentary represents additional considerations for the inspector during the collection of data. From this information, the inspector can complete the element level evaluation. Additionally, element level Smart Flags (Defect Flags) are used to describe a condition which is not included in the National Bridge Element or Bridge Management Element condition state language.

Feasible actions are those that an agency may take to remove the defect. They represent a set of responses that may be taken for an element based upon quantities within a given condition state. They also represent general guidance on agency preservation strategies and can be customized by each agency for each element and condition state.

A summary of feasible actions and associated condition states is given below in Table B.1. Depending on the element, some feasible actions/conditions states may not be available. Other feasible actions, such as "Do Nothing", are available for all element and condition states. "Do Nothing" can be used for all the elements in condition states since the possibility of nothing that needs to be done due to the condition of the element being good or to be used if the condition of the bridge is so severe, the bridge is closed and or there is a feasible action already taking place.

Table B.1 Feasible Actions Depending on Condition States

Feasible Action	Condition State 1	Condition State 2	Condition State 3	Condition State 4
Do Nothing	X	X	X	X
Protect	X	X	X	X
Preserve (for other culverts and other railings)	X	X		
Repair		X	X	X
Rehab			X	X
Reset (for bearings only)			X	X
Replace			X	X

The *AASHTO Manual for Bridge Element Inspection*, *1st Edition*, is useful for developing this information. Some states may provide more detailed information; the following example is from *Bridge Element Inspection Manual*, published by Illinois Department of Transportation, Springfield, Illinois, 2014.

B.2.1 Reinforced Concrete Bridge Elements

Condition State	Condition State Description	Feasible Actions
1	The element shows no deterioration. There may be discoloration, efflorescence, and/or superficial cracking but without effect on strength and/or serviceability. No rust stains visible.	Continue preventive maintenance treatments
2	Minor cracks & spalls may be present but there is no exposed main reinforcing or surface evidence of reinforcement corrosion. Minor rust stains visible.	Continue preventive maintenance treatments and seal cracks and perform non-structural patching if necessary
3	Delaminations and/or spalls in the cover concrete may be present and main reinforcing may be exposed. Corrosion of main reinforcement may be present but loss of section is incidental. Working flexural cracks, map cracked areas or leaching cracks, and other miscellaneous cracks with medium rust staining. Minor impact damage may have occurred.	Clean reinforcement and patch, (and/or seal cracks) Rehabilitate unit
4	Advanced deterioration. Measurable section loss of main reinforcement and/or loss of core concrete section. Internal failures have caused excessive movement. Damages due to vehicular collision that have caused reinforcement to be severed and/or large concrete section loss. Temporary supports may have been installed following structural analysis to allow continued utilization of the structure.	Rehabilitate, strengthen, or replace the element

B.2.2 Steel Substructure Elements

Condition State	Condition State Description	Feasible Actions
1	• Suspected locations where minor fatigue, out-of-plane bending cracks, tears or perforation could develop in structural elements that should be monitored during future inspections.	Continue preventive maintenance treatments
2	• Minor fatigue, out-of-plane bending cracks, tears, or perforation may be present in structural elements. • Minor impact damage may have occurred. • Excessive live load deflection occurs. • Minor pitting has been cleaned and painted that does not require immediate action. Continue preventive maintenance treatments	Grind cracks or plate, lubricate hinges Straighten or install new section Spot paint
3	• Moderate fatigue, out-of-plane bending cracks, tears, or perforation in structural elements parallel to the direction of stress. • Hinges may be frozen from corrosion. • Impact damage has occurred and\or permanent deformations may be evident. • Section loss up to 1/16” per face has been cleaned and painted • Straighten or install new section	Grind cracks or plate; lubricate hinges Straighten or install new section
4	• Severe fatigue, out-of-plane bending cracks, impact damage, tears or perforations have occurred that requires repairs. • Hinges demonstrate uneven alignment horizontally from below or vertically from the deck above. • Temporary supports may have been installed following structural analysis to allow continued utilization of the structure. • Section loss greater than 1/16” per face has been cleaned and painted	Install new sections Replace entire unit

B.2.3 Timber Substructure Elements

Condition State	Condition State Description	Feasible Actions
1	• Investigation indicates no decay • There may be superficial cracks, splits and checks having no effect on strength or serviceability. • Preservative is in place and functioning or untreated timber is sheltered from moisture and not in contact with the earth.	Continue preventive maintenance treatments
2	• Decay, insect infestation, splitting, cracking, checking or minor crushing may exist but none is sufficiently advanced to affect serviceability of the element. • Preservative is in place but demonstrates thinning beyond surface discoloration. • Minor leaking is occurring on untreated sheltered timbers. • Fire damage limited to surface scorching with no measurable section loss.	Continue preventive maintenance treatments Apply preservative where necessary Review moisture protection
3	• Decay, insect infestation, splitting, cracking or crushing has produced loss of strength of the element but not of a sufficient magnitude to affect the serviceability of the bridge. • Preservative is ineffective or nonexistent and timber is exposed to the weather or in contact with the earth. • Fire damage limited to surface charring with minor, measurable section loss.	Apply preservative and/or fumigate Replace affected members
4	• Advanced deterioration. • Decay, insect infestation, splits, cracks or crushing has produced loss of strength that affects the serviceability of the bridge. • Major fire damage which will substantially reduce the load carrying capacity of the member.	Replace affected members Replace entire unit

B.2.4 Precast Concrete Substructure Elements

Condition State	Condition State Description	Feasible Actions
1	• The element shows no deterioration. There may be discoloration, efflorescence, and/or superficial cracking but without effect on strength and/or serviceability. • No rust stains visible.	Continue preventive maintenance treatments
2	• Minor cracks & spalls may be present and there may be exposed reinforcing with no evidence of corrosion. • There is no exposure of the prestress system. • Minor rust stains visible.	Continue preventive maintenance treatments Seal cracks and minor patch
3	• Delaminations and/or spalls may be present. • There may be minor exposure but no deterioration of the prestress system. • Corrosion of non-prestressed main reinforcement may be present but loss of section is incidental. • Minor impact damage may have occurred. • Working flexural cracks and other miscellaneous cracks with medium rust staining.	Clean steel and patch (and/or seal cracks) Rehabilitate unit
4	• Delaminations, spalls and corrosion of non-prestressed main reinforcement are prevalent. • There may also be exposure and deterioration of the prestress system (manifested by loss of bond, broken strands or wire, failed anchorages, etc.). • Damages due to vehicular collision that may have caused wires to be severed and/or large concrete section loss. • Rocking of members may be visible with the passage of live load. • Temporary supports may have been installed following structural analysis to allow continual utilization of the structure.	Rehabilitate unit Replace unit or entire member

B.2.5 "Other" Substructure Elements

Other substructure elements include: stone, masonry, etc. substructure elements

Condition State	Condition State Description	Feasible Actions
1	• There is little or no deterioration • Only surface defects are in evidence	Continue preventive maintenance treatments
2	• There may be minor deterioration, cracking and weathering. • Mortar in joints may show minor deterioration	Continue preventive maintenance treatments Rehabilitate affected area Minor repair, i.e. tuck point
3	• Moderate to major deterioration and cracking. • Major deterioration of joints.	Rehabilitate affected areas Minor repair, i.e., tuck point
4	• Major deterioration, splitting, or cracking	Rehabilitate unit Replace unit

B.3 Comparison of Non-Destructive (NDT) Methods Table

The following table compares NDT methods used for different materials. The table also provides information regarding the effectiveness of the method. Information regarding the time required for use, cost of use, and complexity of use is included. This table was recreated from FHWA NHI Course ILT 130099A Bridge Inspection Non-Destructive Evaluation Showcase (BINS).

Note that the following apply for Table B.2:

C: Concrete

S: Steel

T: Timber

A: All

H: High Effectiveness

M: Medium Effectiveness

L: Low Effectiveness

Table B.2 Summary of NDT Methods

	Cracking	Scaling	Spalling	Delamination	Reinforcement Corrosion	Chemical Attack	PS Strand Corrosion /Breaks	Voids in Grouted Ducts	Surface-Breaking Cracks	Subsurface Cracks	Internal Voids	Porosity and Slag in Welds	Corrosion/Section Loss	Surface Decay and Rot	Internal Decay and Voids	Weathering and Abrasion	Chemical Attack	Distortion /Movement	Time to Use	Cost of Use	Complexity
Material (**C**oncrete, **S**teel, **T**imber, **A**ll)	**C**	**C**	**C**	**C**	**C**	**C**	**C**	**C**	**S**	**S**	**S**	**S**	**S**	**T**	**T**	**T**	**T**	**A**	-	-	-
Visual Inspection	M	M	M	L	L	-	-	-	M	-	-	-	L	M	-	M	L	H	Low	Low	Low
Hands-On Inspection	H	H	H	L	L	-	-	-	H	-	-	L	H	M	-	H	L	H	Med	Low	Low
Borescope	-	-	-	-	H	-	H	H	-	-	-	-	M	-	H	-	-	H	High	Med	Med
Dye Penetrant	-	-	-	-	-	-	-	-	H	-	-	-	-	-	-	-	-	-	High	Low	Low
Ultrasonic Testing – Straight Beam	-	-	-	-	-	-	-	-	H	H	H	M	H	-	-	-	-	-	Med	Med	Med
Ultrasonic Testing – Angled Beam	-	-	-	-	-	-	-	-	H	H	H	H	H	-	-	-	-	-	Med	Med	Med
Ultrasonic Testing – Phased Array	-	-	-	-	-	-	-	-	H	H	H	H	H	-	-	-	-	-	Med	High	High
Acoustic Emission	-	-	-	-	-	-	H	-	H	H	-	-	L	-	-	-	-	-	V. High	V. High	V. High
Magnetic Particle Testing	-	-	-	-	-	-	-	-	H	L	L	-	-	-	-	-	-	-	Med	Low	Med
Eddy Current	-	-	-	-	-	-	-	-	H	L	L	L	-	-	-	-	-	-	Med	Low	Med
Radiographic Testing – Gamma Ray	M	-	-	-	-	-	H	H	M	M	H	H	M	-	L	-	-	-	High	V. High	High
Radiographic Testing – X-ray	M	-	-	-	-	-	H	H	M	M	H	H	M	-	L	-	-	-	High	V. High	High
Chain Drag and Hammer Sounding	-	-	-	M	-	-	-	-	-	-	-	-	-	-	L	-	-	-	Med	Low	Low
Impact Echo	H	-	-	H	M	L	-	H	-	-	-	-	-	-	L	-	-	-	High	Med	High
Ultrasonic Surface Waves	H	-	-	-	M	L	-	H	-	-	-	-	-	-	-	-	-	-	High	High	High
Ultrasonic Pulse Velocity / Pulse Echo	H	-	-	-	M	L	-	-	-	-	-	-	-	-	H	L	-	-	Med	Med	Med
Pachometer / Covermeter	-	-	-	-	L	-	-	-	-	-	-	-	-	-	-	-	-	-	Low	Low	Low
Magnetic Flux Leakage	-	-	-	-	L	-	H	L	H	L	L	-	L	-	-	-	-	-	Med	Med	Med
Ground Penetrating Radar	M	-	M	H	-	-	-	M	-	-	-	-	-	-	-	-	-	-	High	High	V. High
Infrared Thermography	-	-	H	H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Med	High	Med
Electrical Continuity and Conductivity	-	-	-	-	H	-	M	-	-	-	-	-	-	M	M	-	-	-	Low	Low	Low
Half Cell Potential	-	-	-	-	H	-	-	-	-	-	-	-	H	-	-	-	-	-	Med	Low	Med
Electrical Resistivity	-	-	-	-	H	-	-	-	-	-	-	-	-	M	M	-	-	-	Med	Low	Med
Galvanostatic Pulse	-	-	-	-	H	-	-	-	-	-	-	-	-	-	-	-	-	-	High	Med	High
Carbonation Testing	-	-	-	-	L	H	-	-	-	-	-	-	-	-	-	-	-	-	Med	Low	Med
Chloride Ion Testing	-	-	-	-	L	H	-	-	-	-	-	-	-	-	-	-	-	-	High	Med	Low
Rebound Hammer Testing	-	-	-	M	-	L	-	-	-	-	-	-	-	-	-	-	-	-	Low	Low	Low
Penetration Testing	-	-	-	L	-	L	-	-	-	-	-	-	-	H	-	M	M	-	Med	Med	Med
Laser Scanning	L	M	M	-	-	-	-	-	L	-	-	-	L	-	-	-	-	H	Med	Med	Med
Optical holography	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	H	High	V. High	High
Exploratory (Endoscope, Demolition)	-	-	-	-	H	-	H	H	-	-	-	-	M	-	H	-	-	H	High	Med	Med
Material Sampling	-	-	-	H	M	H	-	M	L	L	L	L	H	H	H	H	H	-	High	High	Med
Laboratory Testing	-	-	-	-	-	H	-	-	-	-	-	-	-	-	-	-	H	-	High	High	High

B.4 Summary OF NDT Methods

This section has been reproduced from NHI course 134062 Bridge Rehabilitation Evaluation & Design Course Participant Workbook, Lesson 4.

B.4.1 Visual Inspection (VI)

Principles

- Visual inspection (VI) includes all unaided inspection/evaluation techniques that use the five senses with only very basic tools (for example flashlights, sounding hammers, tape measures, plumb bob, etc.).
- The Aging Aircraft Nondestructive Inspection Validation Center (AANC) defines Visual Inspection as a process of examination and evaluation of systems and components by use of sensory systems aided only by such mechanical enhancements to sensory input as magnifiers, probes, and the like. The inspection process may be done using such behaviors as looking, listening, feeling, smelling, shaking, and twisting. It includes a cognitive component wherein observations are correlated with knowledge of the structure and a general body of information from literature and past experience.

Factors affecting VI

- The accuracy of VI is influenced by four factors: Subject factors, physical factors, environmental factors, and organizational factors. Aspects of each factor are elaborated below.
- *Subject Factors:* Visual acuity, color vision, eye movement, age, experience, personality, intelligence, training, etc.
- *Physical and environmental factors:* Lighting, aids (magnifiers, overlays, viewing screen, CCTV, automatic scanner, etc.), background noise, workplace design, etc.
- *Task Factors:* Inspection time, pace, density of observations, spatial relationship of observations, fault probability, object complexity, etc.
- *Organizational Factors:* Number of inspectors, instructions, supervision, training, standards, budget, alertness, social factors, motivation, incentives, etc.

Advantages

- Visual inspection is inexpensive and easy to execute.

Disadvantages

- A study by FHWA's NDE Validation Center found that fear of traffic, near visual acuity, color vision, formal bridge inspection training, light intensity, accessibility (heights and use of specialized equipment), structure complexity, wind speed, temperature, and

expected time to complete task were factors that negatively affected visual inspection accuracy.

- Further, visual inspection is not likely to detect and identify specific types of defects for which in-depth inspections are prescribed. For example, inspectors charged with in-depth fatigue inspections are not likely to find small cracks. The NDEVC study found that inspectors don't always view welds from the proper distance and are not using appropriate enhancement tools such as lights and magnifiers.

Figure B.1 Visual Inspection Examples

B.4.2 Metal Detection in Concrete: Locate reinforcing and post tensioning tendons

Types of Devices to Detect Metal in Concrete

Rebar location devices are frequently referred to as *covermeters*, *R-meters*, or *pachometers*.

These devices operate on the principle of electromagnetic induction. Unlike concrete, steel bars interact strongly with low-frequency electromagnetic waves applied to the surface of the concrete structure, making it easy to identify their location.

Commercial covermeters either operate on the principle of magnetic reluctance or eddy currents.

Principles of Operation

- Reinforcing bar is detected by a covermeter when the bar lies within the zone of influence of the covermeter search head (yoke or coil). The covermeter response is at a maximum when the search head lies directly over the reinforcing bar. When the bar is not positioned directly under the search head, it can still be detected by the covermeter depending on the magnitude of the horizontal offset. The horizontal offset is defined as the horizontal distance from the center of the bar to the center of the search head.
- Discernment of individual reinforcing bars - A covermeter with a small zone of influence can discern individual bars that are closely spaced with its focused search head. However, these "focused" covermeters also have less penetrating ability and are not able to locate bars that are deeply embedded. Therefore, with this tradeoff recognized, it is desirable to use a covermeter with properties tailored for the specific application.
- Detection of the end of rebar - The zone of influence also affects how accurately the end of a reinforcing bar can be detected. A smaller more focused zone of influence will allow more accurate detection of the end, providing a better estimate of its location.
- Directional Characteristics of Search Heads - Some covermeter search heads can have directional characteristics. For magnetic reluctance covermeters, the shape of the yoke makes it directionally dependent compared to the symmetric coil in an eddy-current meter. For the magnetic reluctance covermeters, the maximum response is found when the yoke is aligned with the longitudinal axis of the reinforcing bar. When testing a structure with an orthogonal gridwork of reinforcing bars, using a magnetic reluctance covermeter would be a benefit if the orientation of the bar needs to be known.

Advantages

- Economical
- Minimal training
- Durable equipment
- Can detect bar size and/or cover, if the other is known

Disadvantages

- Does not have good resolution in areas of high rebar concentration
- Requires calibration or physical verification testing if used to detect bar size and/or depth
- Overestimation of capability

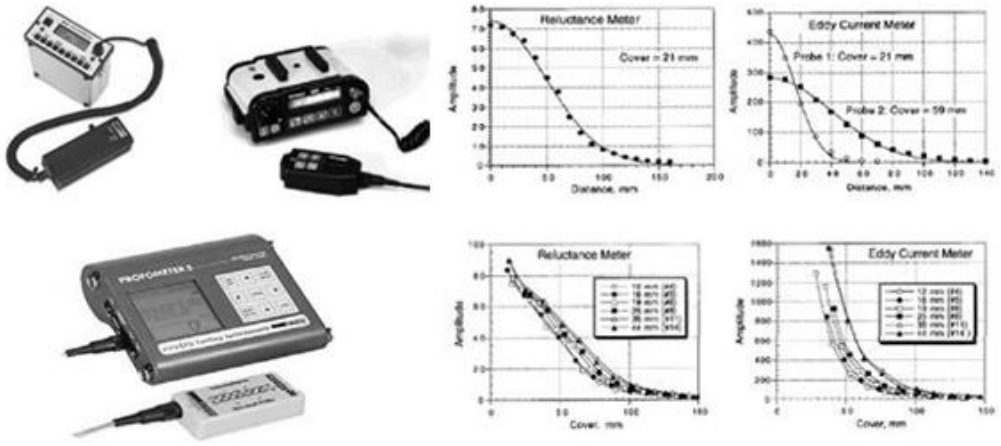


Figure B.2 Examples of Cover Meter Equipment (left) and Cover Meter Output (right)

B.4.3 Sounding: Detection of Delaminations in Concrete

Principles of Sounding (ASTM D4580-03)

- When impacted with a sounding device consisting of steel wheels, chain, or hammer non-delaminated concrete will produce a clear ringing sound. A dull or hollow sound is emitted when delaminated concrete is encountered.
- Bridge decks should be systematically surveyed by delineating the survey area into reasonable spaced grids ranging from 15 in. to 3 ft. on center. The delaminations found within each grid are recorded on scaled drawings of the bridge deck, and an estimate of the total percentage deck delamination calculated.

Typical Procedures

- Electro-mechanical sounding devices employ an electric powered tapping device, sonic receiver, and recorder mounted on a cart to detect delaminations. These devices can produce printed records of delamination surveys. The tapping device consists of steel wheels which impact the surface at 33 times/sec and a piezo-electric hydrophone located in oil filled tires that travel next to the impacting wheels. Calibration of the device is required.
- Chain drag sounding consists of dragging a chain, or tapping the surface with a hammer or rod, and listening for a dull or hollow sound, which denotes a delamination. Chains of 1 in. x ½ in. in diameter have proved effective, either supported from a bar or swung across the deck in longer lengths. Heavier chains may be required under heavy traffic conditions. Quantity of delaminations is recorded using tape measures or estimates.
- Rotary percussion devices consist of toothed-wheels that are rolled across the surfaces, emitting a dull or hollow sound when passed over a delamination. This device can be used on vertical and overhead surfaces.

Advantages

- Easy to perform, with minimal training and at a low cost
- Used to rapidly detect delaminations up to about 5 in. in depth, and more typically in the 1 to 2 in. depth range
- Can be used to detect debonding between PC overlays and deck, but may not differentiate delaminations in substrate.

Disadvantages

- Traffic control is required
- Hearing ability of operator will affect accuracy of soundings
- Not applicable to bridge decks with bituminous overlays
- Traffic noise, fear of traffic, safety of inspector, and time allotted for task each affect accuracy of sounding surveys
- Cannot detect deep or frozen delaminations

Figure B.3 Sounding Examples

B.4.4 Windsor Probe: Uniformity in Concrete Strength and Quality

Principles (ASTM C 803)

- The Windsor Probe uses a powder charge to drive a steel probe into hardened concrete. The depth of penetration is an indication of concrete strength.
- The test has applicability in determining the uniformity in concrete strength and quality. It can be used to obtain an approximate value of compressive strength.

Typical Procedures

The Windsor probe is a penetration test that provides an indication of the compressive strength of a concrete sample. The test consists of driving a .25 in. diameter by 3.125 in. long probe into concrete by means of a precision powder charge. The depth of penetration provides an indication of the compressive strength of the concrete. Although the manufacturer provides calibration charts, the instrument should be calibrated for the type of concrete and the type and size of aggregate used.

Equipment is comprised of a power-actuated gun or driver, hardened alloy probes, loaded cartridges, a depth gauge for measuring penetration of probes, and other related equipment.

Probe test results vary and are not expect to give accurate values of concrete strength. The ability to check the quality and maturity of in-situ concrete is the test's greatest attribute. It provides a means of assessing strength development with curing.

The method has been estimated to be accurate with 5% +/- when the procedure is properly performed and a correlation to the subject concrete is developed.

Advantages

- Identifies questionable areas of in-situ concrete for further investigation
- Fast and simple method of analysis
- Relative quality indicator
- Moderate cost
- Fairly good correlation with other methods

Disadvantages

- Calibration of probe depth to core sample compression strength is required for reliable results
- Results are approximate to near-surface concrete
- Results are affected by surface conditions and aggregate
- Minimum surface damage

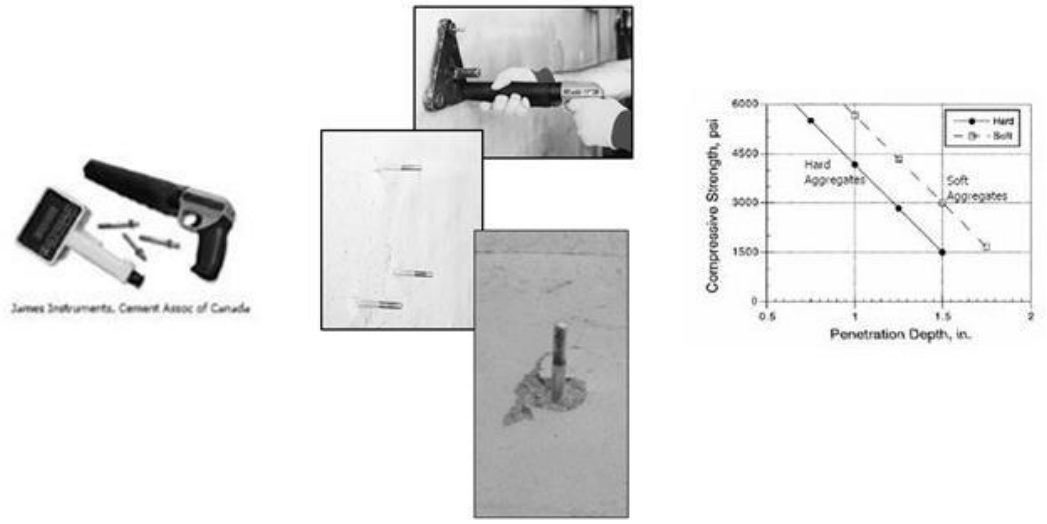


Figure B.4 Examples of Windsor Probes and Resulting Data

B.4.5 Rebound Hammer: Uniformity in Concrete Strength and Quality

Principles (ASTM C 805)

The rebound hammer, typically called Schmidt Hammer, is a surface hardness test method. The rebound principle consists of a spring-driven mass that is driven against the surface of a concrete specimen with known energy. The rebound distance is measured and the "hardness" of the surface is estimated from this value. The energy absorbed by the specimen is related to both its strength and stiffness.

The accuracy of the rebound hammer has been estimated between 15-20% in laboratory testing and +/- 25% in field testing.

Typical Procedures

- Rebound hammer can be used in horizontal, vertical or inclined position.
- Hammer must be perpendicular to surface being tested.
- The surface is impacted with a steel probe and the rebound is measured from 10 to 100. The greater the rebound number the higher the concrete stiffness, and hence, quality.
- Correlation with concrete under investigation is required to validate test results, although a calibration curve is provided with the instrument which can be used to approximate strength.

Advantages

- Easy and economical to use
- Provides rapid means of assessing concrete quality, and identifying areas for further investigation.

Disadvantages

- Provides near surface measurement of concrete properties and consequently may be influenced by surface conditions, such as smoothness, rigidity, moisture content, and density (carbonation).
- Aggregate quality and quantity can influence results.
- Should not be used on early age concrete.
- Requires calibration to core sample tests to validate results.

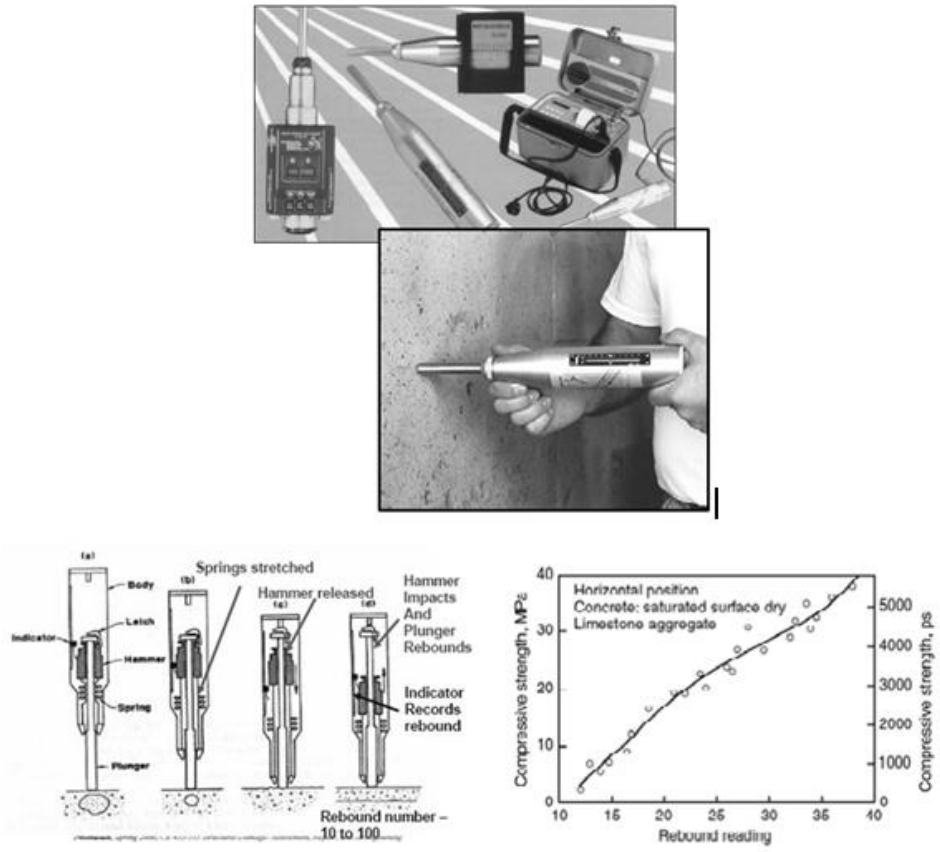


Figure B.5 Rebound Hammer Examples of Equipment and Data Output

B.4.6 Impact Echo: Voids and Delaminations in Concrete Structures ASTM C1388

Principles of Impact Echo

In the impact-echo method (IE), an impact generates stress waves which propagate in all directions. The duration of the impact is typically less than 100 microseconds. A transducer next to the point of impact monitors surface motion and records the waveforms. The waves are reflected by internal interfaces or external boundaries. When the echoes return to the surface, a transducer with a data-acquisition system records their displacements. The frequency of the surface response is analyzed to determine depth and location of defect. Analysis depends on wave speed in the test specimen. Calibration is required.

Typical Applications

- Determination of thickness of plate-like structures
- Detection of damage (voids, flaws or honeycombs) in slabs and bridge decks with or without overlays.
- Location of voids in beams, columns, grouted tendon ducts and hollow cylindrical members, to assess the quality of the bond in overlays at the asphalt-concrete interface and at reinforcement, and to measure the depth of surface opening cracks.

Wave speed provides an indicator of the quality of the concrete. The average wave speed in concrete is approximately 3660 m/sec (12,000 ft./sec). A wave speed lower than this may indicate poor quality.

Advantages

- Access to one surface is sufficient to determine depth and extent of defect
- Minimal, relatively inexpensive equipment that is practical for use in the field
- Does not damage member being evaluated

Disadvantages

- Data interpretation/analysis by experienced operator required
- Complex waveforms due to stress wave propagation in all directions complicates analysis
- Heavy reinforcement and/or multiple boundary conditions complicates waveforms
- Limited to a depth of about 1 meter
- Expensive for assessment of large areas

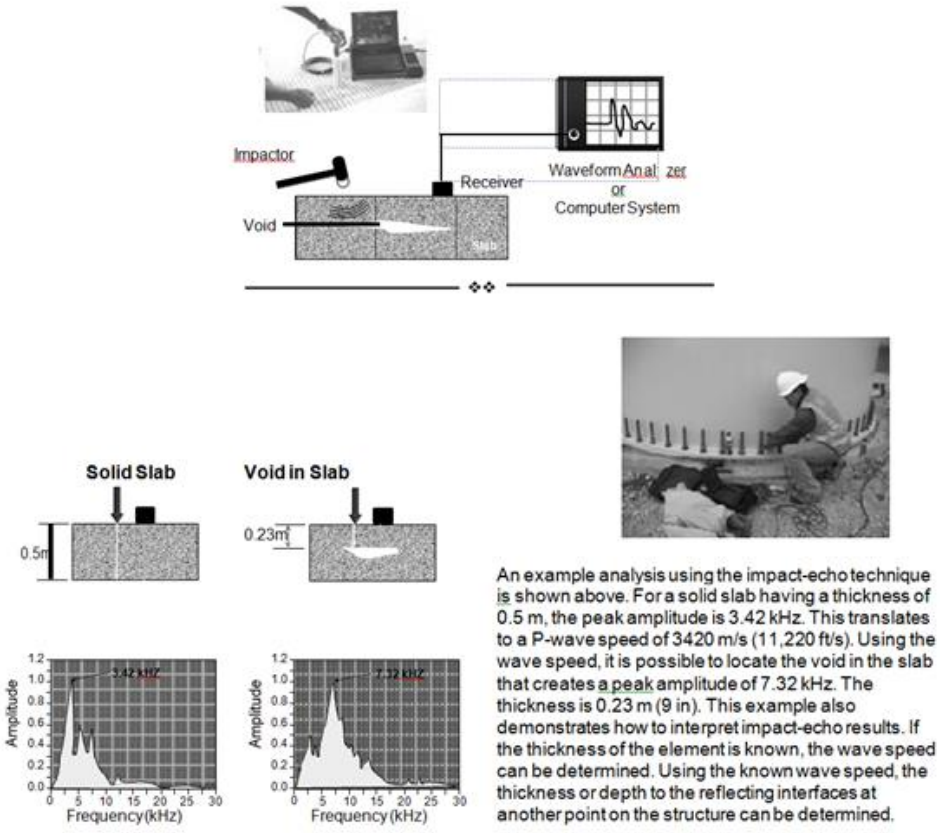

Impactor
Receiver
Waveform Analyzer or Computer System
Void
Slab
Solid Slab
Void in Slab
0.5m
0.23m
Amplitude
Frequency (kHz)
An example analysis using the impact-echo technique is shown above. For a solid slab having a thickness of 0.5 m, the peak amplitude is 3.42 kHz. This translates to a P-wave speed of 3420 m/s (11,220 ft/s). Using the wave speed, it is possible to locate the void in the slab that creates a peak amplitude of 7.32 kHz. The thickness is 0.23 m (9 in). This example also demonstrates how to interpret impact-echo results. If the thickness of the element is known, the wave speed can be determined. Using the known wave speed, the thickness or depth to the reflecting interfaces at another point on the structure can be determined.

Figure B.6 Impact Echo Schematic, Setup, and Example Output Data

B.4.7 Ground Penetrating Radar (GPR)

Radar uses electromagnetic waves to detect subsurface anomalies. An antenna, generally affixed to a vehicle, is used to transmit short pulses of electromagnetic energy into the surveyed material. A portion of the energy is reflected back to the antenna when an interface between materials of dissimilar dielectric properties is encountered. The antenna then generates an output signal that contains information on what was reflected, how quickly the signal traveled, and how much signal was attenuated.

Typical Applications

Radar uses electromagnetic waves to determine concrete thickness, locating voids and rebars, and identifying deterioration.

Advantages

- Assess greater area than other methods, saving time and costs
- Access to only one side required
- Very sensitive to presence of embedded metal objects
- Very sensitive to presence of water
- Can penetrate concrete up to 12 to 16 inches, although resolution decreases with depth
- Provides record of inspection
- Can “see” below overlays

Disadvantages

- Extensive data interpretation/analysis, which requires experience
- Expensive equipment
- Environmental conditions can affect testing
- Congested reinforcement can mask deficiencies
- Difficult to penetrate through wet substrates; wet chlorides can skew results
- Cracks, delaminations, and honeycombing are difficult to detect unless moisture present
- Requires skilled operator and calibration

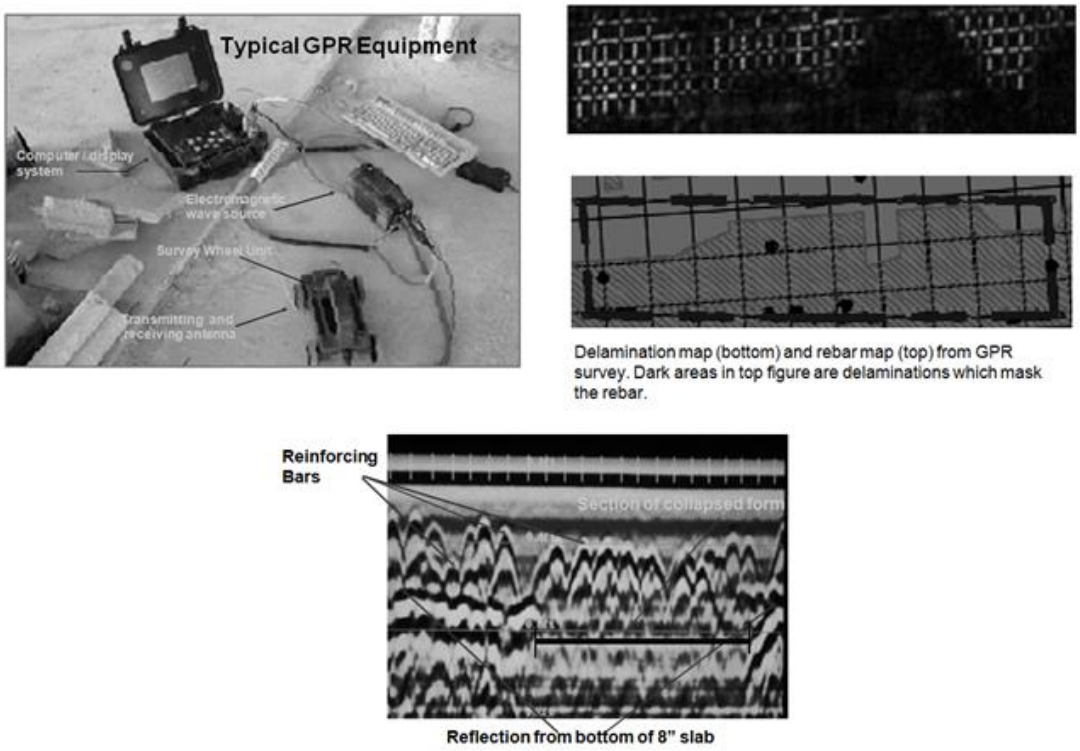

Typical GPR Equipment
Computer / display system
Electromagnetic wave source
Survey Wheel Unit
Transmitting and receiving antenna
Delamination map (bottom) and rebar map (top) from GPR survey. Dark areas in top figure are delaminations which mask the rebar.
Reinforcing Bars
Section of collapsed form
Reflection from bottom of 8" slab

Figure B.7 GPR Equipment, Grid, and Output

B.4.8 Ultrasonic Pulse Velocity: Evaluation of Concrete Structures

Principles of UPV (ASTM C597)

The ultrasonic through transmission method is commonly known as the Ultrasonic Pulse Velocity (UPV). In this method, a stress pulse of compressive waves travels through the mature concrete, and the travel time is measured for a known path length. The UPV can then be calculated. An ASTM standard has been written detailing pulse velocity testing in concrete, ASTM C 597 "Standard Test Method for Pulse Velocity Through Concrete".

The pulses of compression waves are generated by an electro-acoustical transducer that is held in contact with one surface of the concrete under test. Waves are received by another transducer, which measures the transit time (T). Transducers with a resonant frequency of 50-kHz are most common.

Typical Procedures

The pulse velocity of ordinary concrete is typically 3660 m/sec (12,000 ft./sec). The speed depends on the concrete's density and stiffness. Changes in the speed are apparent in concrete that has not been uniformly consolidated, cured, or has different materials or mix proportions. Based on these reasons, the factors that affect pulse velocity for a concrete of a given strength are the aggregate type, aggregate content, moisture content, and the presence of reinforcement. Different aggregate types have different moduli. An increase in aggregate content means the wave travels through a denser material. The wave also travels faster through water than air. Steel increases the pulse velocity (1.5 to 2 times higher) when it is in the path of the pulse. There is no simple correlation between PV and concrete properties, like compressive strength.

When a defect is present, like an air void, the travel time increases, i.e., the velocity decreases. This is shown schematically in Figure B.8. For path 1, the velocity is the fastest. It travels through good quality concrete. For path 2, the velocity decreases as the wave has to travel through inferior concrete. For path 3, the wave has to travel around the crack, and the velocity will be less than it was for path 1. For path 4, the wave cannot be detected by the receiver. The air interferes with its transmission. A comparison of the measured pulse velocities provides an indication of the relative quality of the concrete.

Important Notes

Wave speed provides an indicator of the quality of the concrete. The average wave speed in concrete is approximately 3660 m/sec (12,000 ft./sec). A wave speed lower than this may indicate poor quality.

Advantages

- Relatively fast
- Ability to test long path lengths (up to 60 ft.)
- Relative indicator of material properties
- Can be correlated to strength/elastic properties

Disadvantages

- Two sided access required
- Accurate path length distance must be known
- Signal strength affected by acoustic coupling
- Wave speed can be influenced by concrete properties and steel

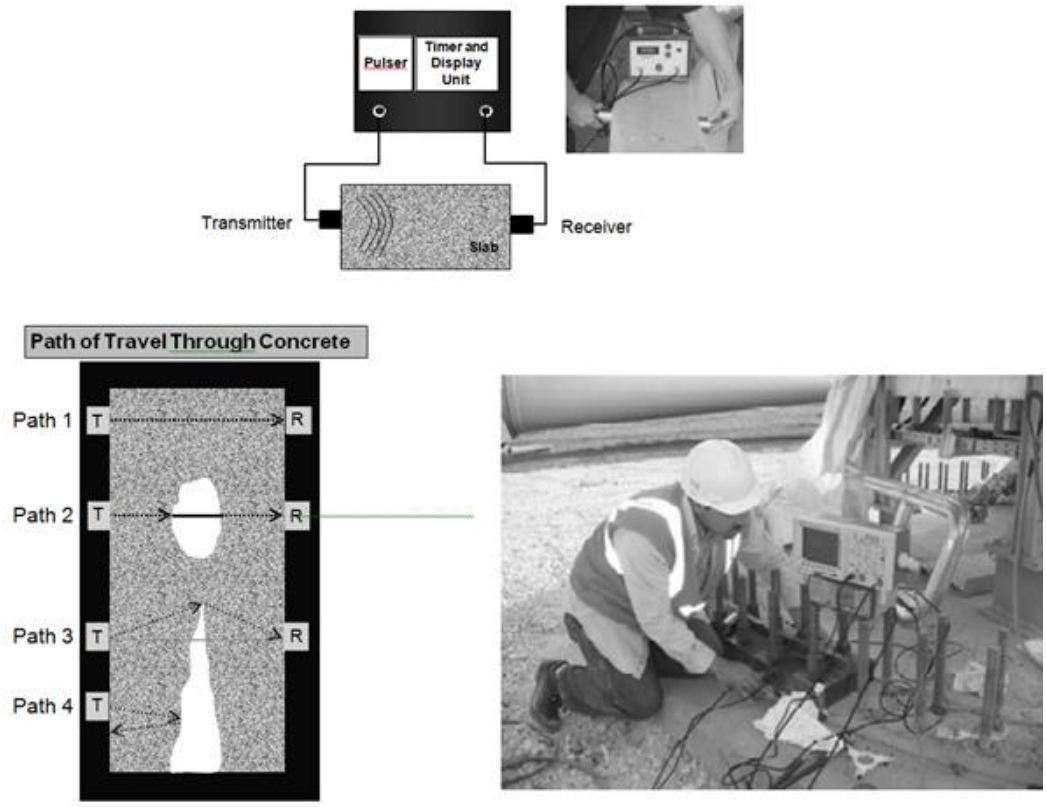


Figure B.8 Ultrasonic Pulse Velocity Method Schematic (top), Wave Path Examples (left), Equipment (right)

B.4.9 Infrared (IR): Detection of Near Surface Anomalies

Principles (ASTM D4788)

- Infrared Thermography is based on the principle that subsurface anomalies in a material affect heat flow through that material. Localized differences in surface temperature cause changes in the heat flow, or emitted radiation. By measuring the electromagnet radiation in the IR spectrum, anomalies can be detected.
- The wavelength of the emitted electromagnetic radiation is proportional to temperature.
- Anomalies associated with poor concrete, namely defects therein, decrease the thermal conductivity of concrete without substantially increasing the convection effects.
- ASTM D 4788 "Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography"
- A temperature gradient is required to detect defects, which requires approximately 4 hrs. of direct sunlight for bare concrete decks and 6 hrs. of sunlight for asphalt covered decks.

Typical Applications

- IR is applicable to the detection of delaminations, voids and cracking in bridge elements, including decks.
- IR can be used on decks with and without asphalt.
- IR can be used to detect debonding and delaminations in carbon fiber composites.

Advantages

- Generally fast, non-contact method of survey allowing economical collection of data on large areas
- Works on wide range of materials, including asphalt overlay bridge decks
- Limited to near surface defects
- Skilled operator required
- Good documentation of inspected areas

Disadvantages

- Depth or thickness of defect cannot be determined
- Requires uniform heating, so shade or wind can effect results
- Moisture can affect results
- Requires trained staff for post-processing of data
- As depth of defect increases, detection ability decreases

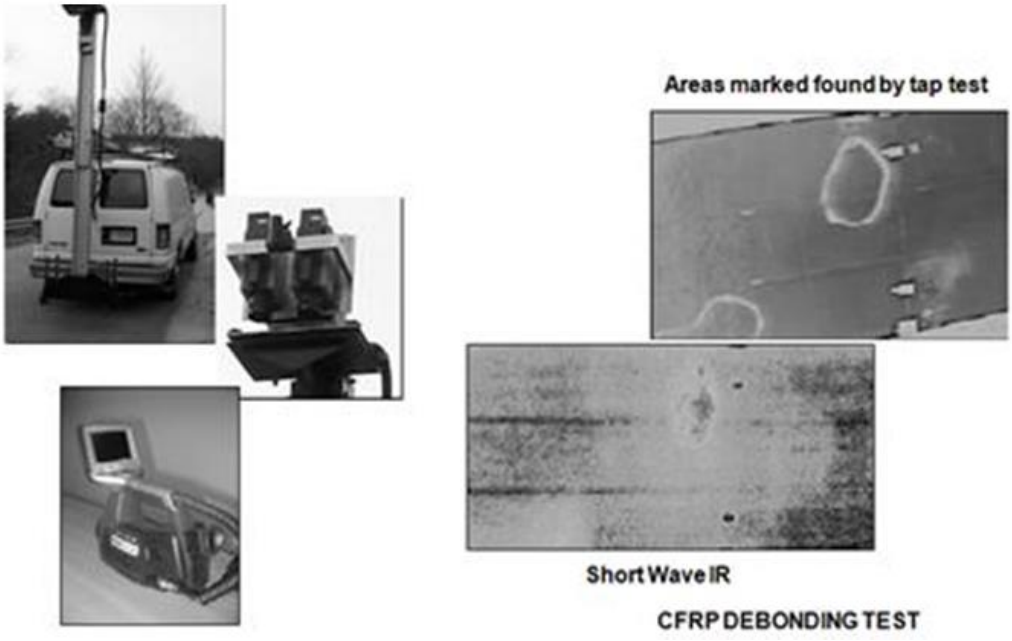


Figure B.9 IR Figures

B.4.10 Petrography: Study of Microscopic Structure of Concrete

Principles

Petrographic examinations can be made on concrete fragments and cores removed from suspect areas. It is best to subject poor and good quality areas to examination to better diagnose the distressed concrete.

The following information can be gained through petrography studies:

Evaluation

- Mix proportions
- Effectiveness of mixing
- Consolidation
- Curing/carbonation Water content
- Degree of maturity (cement hydration)
- Bonding within concrete matrix
- Discoloration/staining

Detection

- Segregation caused by over vibration
- Course ground cement
- Partially hydrated or old cement
- Retempering
- Supplemental cementitious materials
- Alkali Silica Reactions (ASR)
- Delayed Ettringite Formation (DEF)
- Freezing of plastic concrete
- Freeze thaw distress
- Aggregate soundness
- Sulfate attack
- Contaminants
- Aggregate composition

Determine

- Air content and parameters of air void system
- Water/ Cement Ratio

Advantages

- Provides insight into existing concrete properties related to durability

Disadvantages

- Can be time consuming
- Requires sample removal
- Results are local, not necessarily representative of global situation

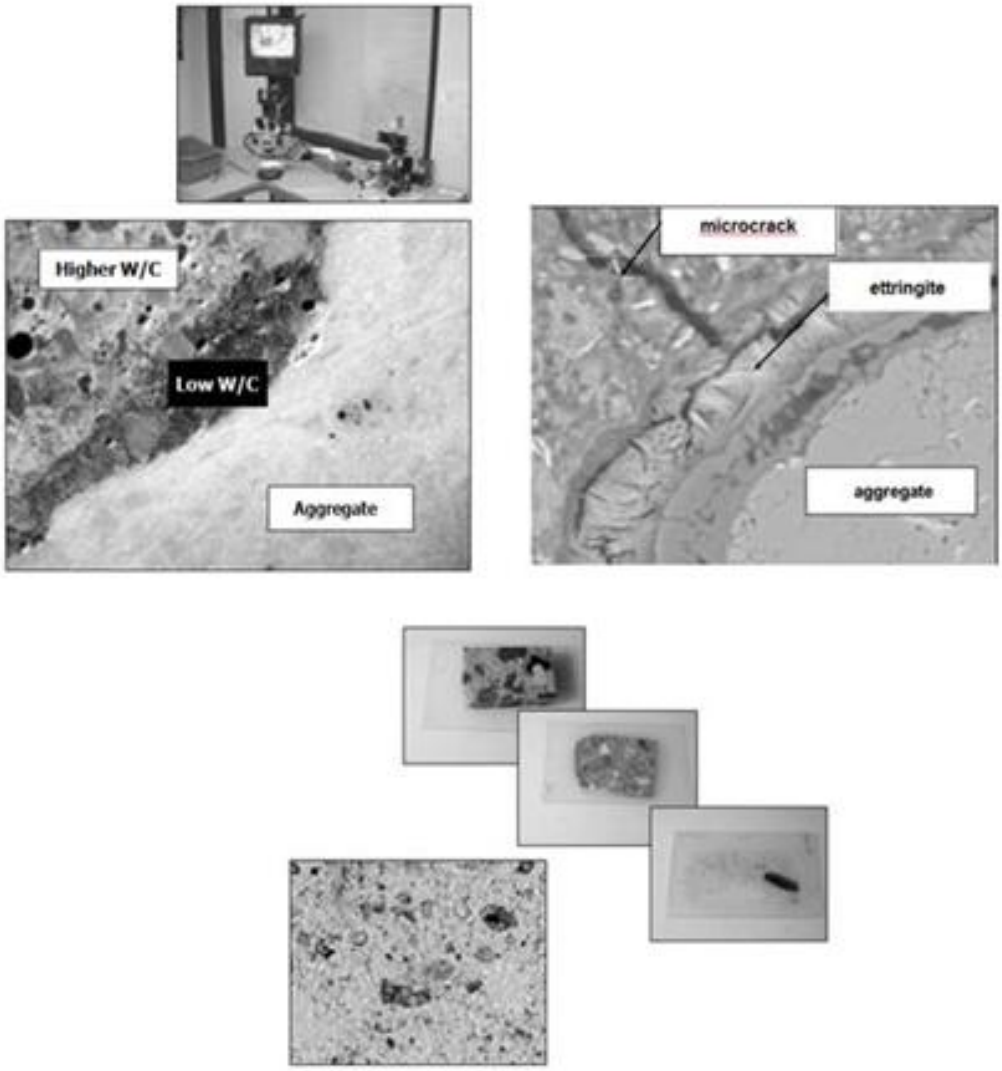


Figure B.10 Petrographic Images

B.4.11 Magnetic Flux Leakage

Magnetic Flux Leakage is the measurement of changes in the path of magnetic force lines, or flux, near a ferromagnetic material, such as steel, with discontinuities or defects. In the presence of a magnetic field, ferromagnetic materials align their electric dipoles with the external field. A stronger field yields more aligned dipoles. There is a saturation point where all dipoles are aligned and no further alignment is possible. The amount of the alignment of these dipoles, and subsequently the flux, depends on the intensity of the applied magnetic field. It is therefore possible to increase the intensity of magnetic flux leakage to overcome some of the inherent limitations in terms of sensitivity, signal-to-noise ratio, etc. The applied magnetic field needs to be large enough to overcome problems due to noise, distance between the magnetic source and ferromagnetic field, and the masking effect of the large quantities of steel typically found in prestressed and reinforced concrete members. When an external magnetic field is applied to reinforced or prestressed concrete members, the flux within the steel remains unchanged until it must leave the steel to travel back to the south pole of the magnet. If the flux encounters a flaw, such as corrosion or broken and fractured steel, the flux will "leak" out at these areas and not travel to the south pole of the magnet. The amount of leakage can be measured using Hall-effect sensors. The crystals of the sensors react to the presence of an external magnetic field when excited by developing a voltage difference across the two parallel faces of the crystal, this is known as the "Hall Effect."

Magnetic flux leakage Magnetic flux leakage (MFL), formerly referred to as magnetic field perturbation testing (MFP), consists of subjecting a magnetically permeable material to a magnetic field. The field strength requirements depend on the permeability of the material and the sensitivity of the test probes. The principles of magneto-statics demonstrate that when a homogeneous magnetically permeable material is immersed in a static, uniform, external magnetic field, the magnetic field within the material approaches the same magnitude as the magnetic field in which it is immersed. These perturbed fields are called leakage fields due to the "leakage" of magnetic flux out of the material and into the air. Leakage is caused by the reduction in cross-sectional area of the magnetic material due to the anomaly. Inspections using magnetic flux leakage techniques The preferred characteristic of the MFL technique is that mechanical contact with the test specimen is not necessary. In many cases, both the exciting coil and the sensing coil may be operated without directly contacting the material. This advantage has particular importance in the structural concrete industry. Another benefit of MFL is that it requires no specific surface preparation other than cleaning. This method is easily automated for high-speed, detailed testing. It is also useful for identifying surface cracks, near-surface inclusions, and nonmagnetic coating thickness on a permeable base, as well as in monitoring physical and mechanical properties that cause changes in magnetic permeability. Sensitivity is limited by ambient noise and magnetic fields. The test specimen must be magnetically saturated, thereby limiting demagnetization concerns. The extreme sensitivity of the method has been utilized to detect surface cracks and subsurface inclusions on the order of 0.3 mm.

The magnetic flux leakage method has been applied to the determination of nonmagnetic coating thickness, the depth of case hardening, and the carbon content of a material.

Another major application has been in testing steel bearing raceways and gear teeth. In its most refined form, MFL is one of the most sensitive methods for the detection of surface and near-surface cracks and flaws in ferromagnetic materials.

B.4.12 Impulse Response: Evaluation of Concrete Structures

Principles of Impulse Response

- Impulse Response is a nondestructive testing technique in which mobility of the test structure is measured. Slab impulse response is used to identify poor support zones below pavements, and to map out the extent of the voids. It is also able to evaluate the effectiveness of repairs to the slab-subgrade, by comparing test results before and after repairs are made. This test was first developed in France to assess the condition of deep pile foundations, but its applications have been expanded to other structures like pavements. The pavements can be either concrete or asphalt, with or without reinforcement.
- To perform an impulse-response test, an operator strikes the surface being tested with an instrumented hammer, generating a stress pulse with a wide range of frequencies. A soft rubber-tipped hammer generates frequencies up to 1000 Hz, and a metal-tipped hammer generates frequencies up to 3000 Hz. The input force is continuously monitored. A geophone (receiver) near the point of impact measures the vibrations of the structure.

Procedures

A plot of the geophone particle velocity divided by the hammer vibration force (v/F), known as mobility, versus frequency is used to assess the condition of the structure. The signals from the hammer and the geophone are sent to a digital oscilloscope and the data is processed in the field using the FFT (Fast Fourier Transform) algorithm.

A test plot can be divided into two portions:

Low frequencies (less than 100 Hz): The initial linear portion of the plot (0 to 50 Hz) yields the slope or the compliance. Its inverse is the dynamic stiffness. This is a property of the structure-soil composite. It can be used to assess the structure on a comparative basis and to establish uniformity.

High frequencies (greater than 100 Hz): When testing piles, this portion of the plot represents their longitudinal resonance and is a function of shaft length and the degree of toe anchorage. The relative amplitude is a function of lateral soil damping. When testing a plate element, the mean mobility (from 100 to 1000 Hz) is related to the density and thickness of the element. Cracking or honeycombing in the element will decrease the stability of the mobility plot from 50 to 1000 Hz, reducing damping of the signal. If delaminations or voids are present, there will be an increase in the ratio of peak mobility to average mobility.

The hammer impact generates only a small amount of energy. Soil damping limits the depth where useful information can be obtained for both piles and pavements. For slabs with a thickness greater than 0.6 m (2 feet), data interpretation becomes difficult. The stiffness of the system is controlled by the pavement and not by the support under the slab. However, even if the high frequency response is not present, the dynamic stiffness can be used to assess the structure.

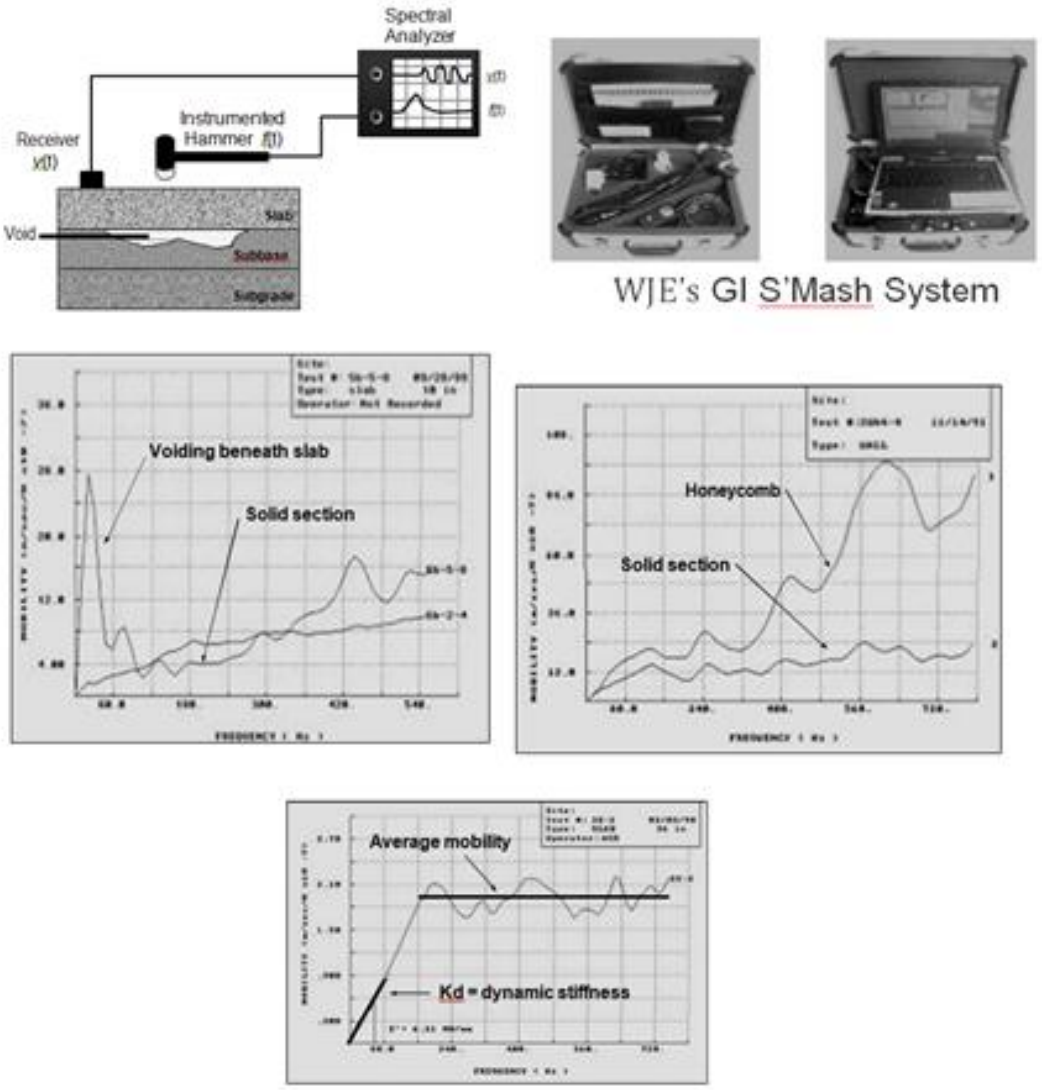


Figure B.11 Impulse Response Systems (top) and Data (bottom)

B.4.13 Half Cell Testing: Corrosion Potential Measurement

Principles of Half Cell Testing (ASTM C876)

During active corrosion electrons are liberated form iron (anodes), which moves into the concrete as ferrous ions (Fe^{2+}). This process is called half-cell oxidation, or the anodic reaction, and is represented as follows:

$$Fe \rightarrow Fe^{2+} + 2e^-$$

The electrons ($2e^-$) move to cathodic sites and combine with water and oxygen present in the concrete, which is represented as follows:

$$2H_2O + O_2 + 4e^- \rightarrow 4OH^-$$

To maintain electrical neutrality, ferrous ions migrate through concrete to cathodic sites where they combine to form hydrated iron oxide, or rust. When the bar is not corroding, there is no flow of electrons or ions.

As ferrous ions move into the surrounding concrete, the electrons left behind in the bar give the bar a negative charge. The half-cell potential method is used to detect this negative charge, and thereby provide an indication of corrosion activity.

During the test, a copper-copper-sulfate ($Cu\text{-}Cu\text{-}SO_4$) electrode is connected to the steel through a high-impedance voltmeter with negative lead attached to the half-cell. If the bars are corroding, electrons will tend to flow from the bar to the half-cell. These electrons are consumed in a reduction reaction, transforming copper ions in the solution into copper atoms deposited on the copper rod. This process produces a measured voltage at the voltmeter. The more negative the reading, the more electrons flowing from the steel, and the greater the reduction reaction.

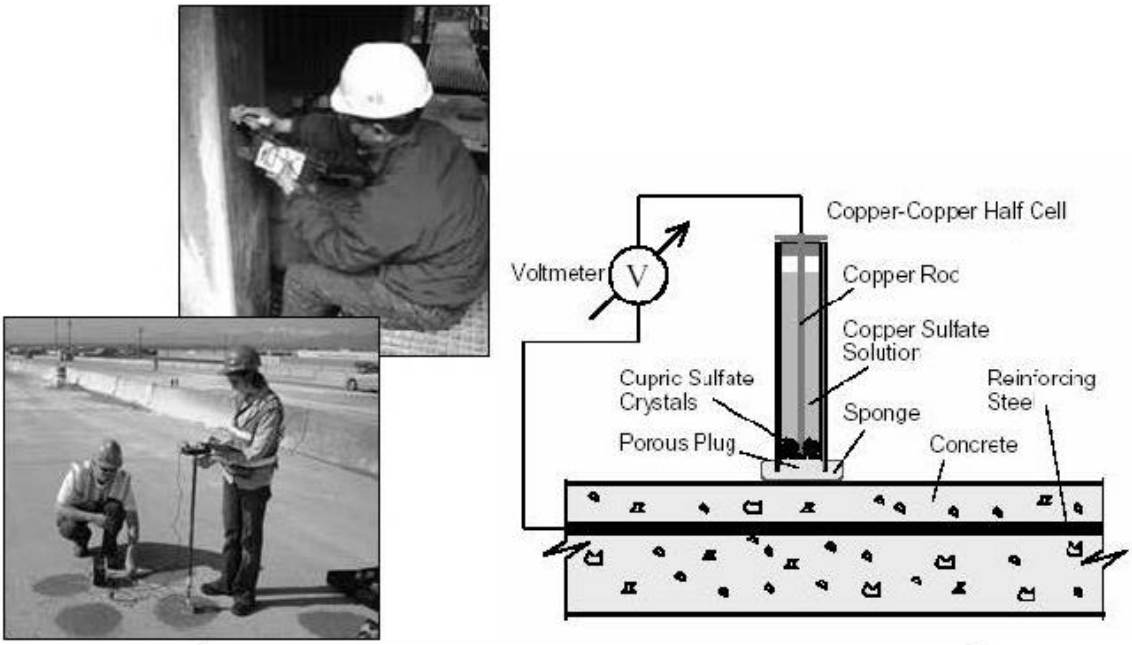
The half-cell potential readings are indicative of the probability of corrosion activity, as shown in Figure B.12.

Advantages

- Identification of areas of potential corrosion
- Lightweight, portable equipment
- Inexpensive and quick

Disadvantages

- Requires direct connection to reinforcement
- Electrical continuity of reinforcement in structure is required, hence not applicable to epoxy coated bars
- Concrete needs to be moist
- The following factors can influence the measured potentials: Concrete resistivity, Temperature, Carbonation, Oxygen content, Rebar coatings, and Degree of moisture saturation



How to interpret half-cell readings?

If HCP > -0.200 V_{CSE}:

Probability is greater than 90% that there is no corrosion activity

If -0.200 V_{CSE} < HCP < -0.350 V_{CSE}:

Corrosion activity is uncertain

If HCP < -0.350 V_{CSE}:

Probability of corrosion is greater than 90%

Figure B.12 Half-Cell Testing Images and Interpretation

B.4.14 Linear Polarization: Rate of Corrosion

Principles of Linear Polarization (ACI 228.2R98, SHRP S324, S330)

The Linear Polarization Method measures the rate of corrosion in reinforcing steels.

The technique measures the current required to change by a fixed amount the potential difference between the reinforcement and a standard reference electrode. The measured current and voltage allow determination of the polarization resistance, which is related to the rate of corrosion.

For a small perturbation about the open circuit potential, a linear relationship exists between the change in voltage and the change in current per unit area of bar surface. This is the polarization resistance, R_P:

$$R_P = \Delta E / \Delta i$$

Where ΔE = change in voltage and Δi = change in current per unit area. There is an established relationship between corrosion rate of the anode and the polarization resistance:

$$i_{corr} = B / R_P$$

Where i_{corr} = corrosion current density, in units of mA per ft^2, B= a constant based on polarization curves (taken as 0.026V for steel in concrete), and R_P = Polarization resistance in Ω/ft^2.

- If $i_{corr} < 0.2$ mA/ft^2, no corrosion damage is expected
- If 0.2 mA/ft^2 < i_{corr} < 10 mA/ft^2, corrosion damage is possible with 10 to 15 years
- If $i_{corr} > 10$ mA/ft^2, corrosion damage is expected within 2 to 10 years

Advantages

- Portable and lightweight equipment
- Determines rate of corrosion at the time of testing

Disadvantages

- Requires direct electrical connection to reinforcement and interconnection of reinforcing internally
- Cannot test decks with epoxy coated or galvanized rebar
- Concrete must be uncracked, even, and free of coatings and visible moisture
- Cover depth must be less than 4 in.
- Results can vary widely across a deck
- Requires knowledgeable personnel to interpret the results

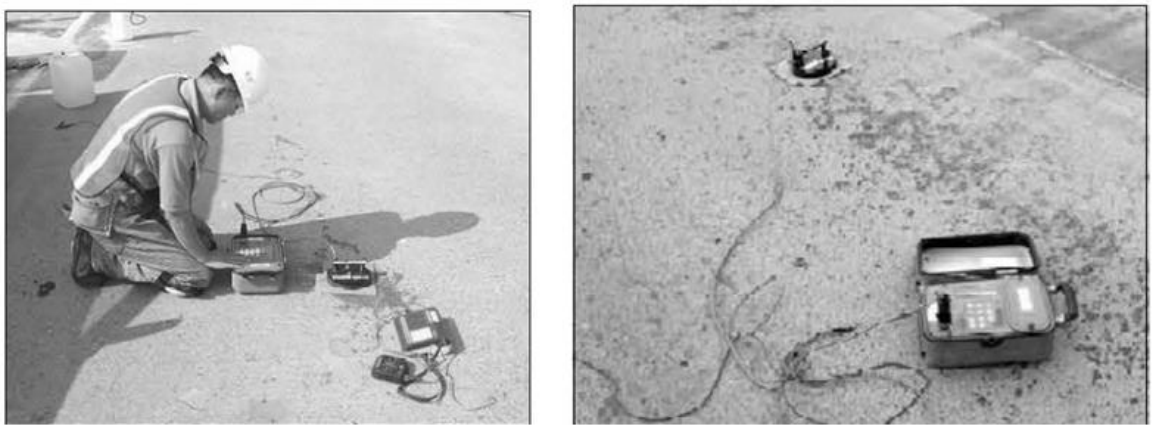


Figure B.13 Examples of Linear Polarization Equipment

B.4.15 Concrete Resistivity

Principles of Resistivity (ASTM D3633)

- The rate at which the corrosion reaction can proceed is governed by the amount of oxygen available to the reaction, the alkaline state of the concrete, and the resistivity of the concrete.
- The electrical resistance of concrete, which depends on the microstructure of the paste, moisture content of the concrete and chloride content, affects the ionic current flow between anode and cathode and hence the rate at which corrosion can occur.
- A high concrete resistivity decreases current flow and impedes the corrosion process
- A highly permeable concrete will have a high conductivity and low electrical resistance. Because resistivity is proportional to current flow, the measurement of the electrical resistance of concrete provides a measure of the possible rate of corrosion.

Typical Procedures

The equipment relies on four equally spaced electrodes placed in drilled holes. The outer probes are connected to an alternating current. The inner probes are connected to a voltmeter. The apparent resistivity, ρ, is given by:

$$\rho = 2\pi sV / I$$

Where s = spacing of probes, V = measured voltage between inner electrodes, and I = current between outer electrodes.

Advantages

- In combination with other tests, such as the half-cell test, resistivity measurements may give an indication of corrosion. Concrete with high resistivity may give an indication of corrosion. Concrete with high resistivity, at locations where half-cell results show corrosion, indicates a very slow corrosion rate.
- Resistivity measurements can also be used to assess permeability.

Disadvantages

- There are no generally agreed upon interpretation guidelines. Resistivity equipment suppliers and Bungey (Bungey, J.H. 1989) recommend:

  >20 kΩ-cm: Low corrosion rate

  10-20 KΩ-cm: Low to moderate corrosion rates

  5-10 kΩ-cm: High corrosion rates

  <5 kΩ-cm: Very high corrosion rates

- Other researchers (Feliu, S. et al., 1996) and Manning (Manning, D.G. 1985) have recommended alternative guidelines.
- Since carbonation seriously affects surface resistance, measurement on the concrete surface should be avoided.

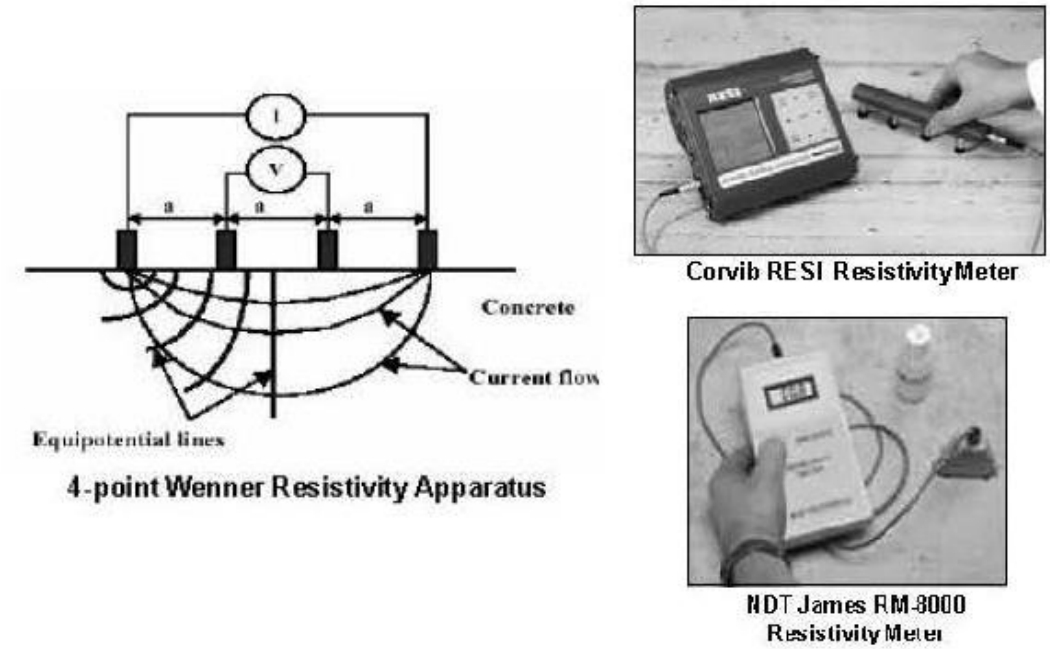


Figure B.14 Resistivity Schematic and Apparatus

B.4.16 Chloride Testing

Principles

- Laboratory tests to determine chloride content include Acid- Soluble Chloride Determination (ASTM C1152), Water-Soluble Chloride Determination (ASTM C1218) and Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials (AASHTO T 260-05).
- Field tests are also available to determine chloride content and utilize a specific ion electrode.
- Chloride sampling should include determination of the chloride profile with depth in the concrete. Chloride samples are collected incrementally from the surface either by taking drillings or sections from cores. The first 0.25 in. is usually discarded as being directly influenced by the immediate environment. For improved statistical accuracy, multiple adjacent drillings are made and the depth increments from each drilling are mixed.
- When powder samples are obtained, drill to depth of 0.5 in. less than desired sample depth. Using a drill bit having 0.25 in. smaller diameter than the initial drill bit, extend hole by 0.5 in. to target depth collecting 10g of powder excavated from hole. Take care not to use contaminated tools, touching side walls, or using hands, which can influence results.
- The chloride extraction methods require that a minimum of 10g of material be obtained and pass through a No. 50 (0.0018 inch) screen. Water or acid is then used to extract chlorides into solution, where they can be measured.

Total (Acid-Soluble) Chloride vs. Water-Soluble Chloride

- Approximately 50% of total chlorides in concrete are tied-up by the cement matrix. Chlorides bound by the matrix are not water soluble, hence acid-soluble test methods are used to provide a better indication of the total chloride content.
- Some aggregates contain chlorides which may become unbound during the pulverization process, and hence are detectable during the water-soluble test.
- Acid soluble testing is more reproducible and less time consuming as compared to the water soluble test method.
- The “threshold” or minimum level of water-soluble chloride contamination in concrete necessary to corrode black bar reinforcing steel is 1.0 to 1.5 lbs. chlorides/yd3, 1 to 2% chloride by weight of cement or 0.02 to 0.03 % chloride by weight of concrete.
- The water-soluble chloride content is about 50% to 80% of the total (acid-soluble) chloride content.

Advantages

- Chloride content results can be used to infer corrosion activity, as well as future corrosion potential if permeability rates are known.

Disadvantages

- Field test methods are not as accurate as laboratory testing, and correction factors are required to calibrate field test methods to laboratory methods.
- Hydroxyls can influence minimum threshold levels for the initiation of corrosion.

- Water soluble test methods do not measure total chloride contents, due to its inability to extract bound chlorides during testing. The detection of bound chlorides, which are freed during aggregate pulverization, can negatively influence test results.

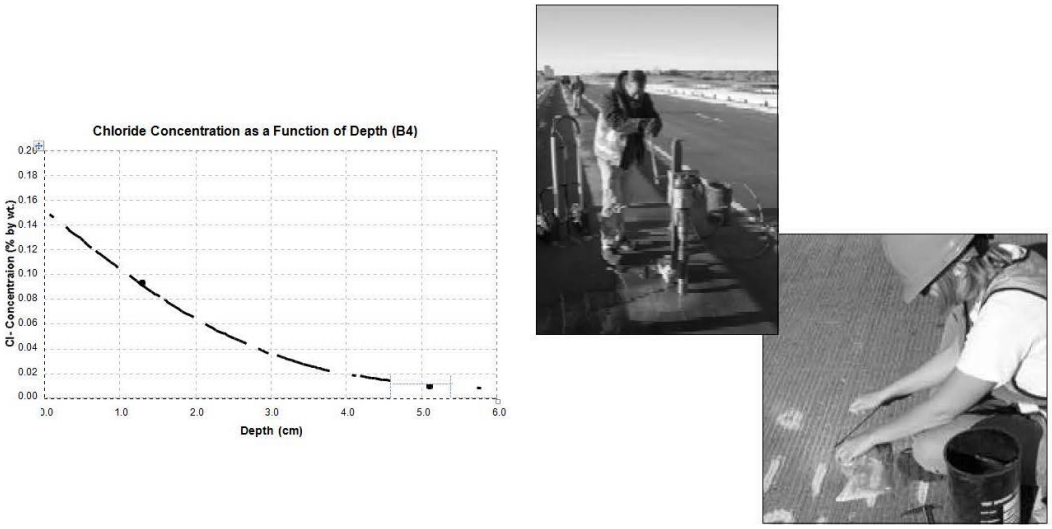


Figure B.15 Chloride Measurement and Example Data

B.4.17 Other Concrete Properties: Permeability & Alkalinity

Permeability

- The permeability of concrete is important in determining the extent to which aggressive substances can attack embedded reinforcement.
- Low permeability concrete generally is of superior strength, and is resistant to water/chloride infiltration. Porous concrete allows water, oxygen, and chlorides to reach rebar, which accelerates corrosion of reinforcing.
- Methods for assessing concrete permeability include Chloride 90-day Ponding Test and Chloride Ion Resistance (Rapid Permeability Test).
- Core samples are removed from the subject structure and returned to the laboratory for testing.
- Permeability testing can be used to measure the relative durability of one concrete verse another, and is generally used during mix design development.

90-Day Ponding Test (AASHTO T-259)

- In this test, water containing a 3 percent sodium chloride solution is ponded on a core sample for a period of 90 days. After 90-days, the concrete is sampled for chloride ion content at various depths.
- Advantage: Test is capable of determining susceptibility of concrete to chloride ion intrusion.
- Disadvantage: Requires a long period to complete the test and analysis.

Rapid Permeability Test (ASTM 1202, AASHTO T-277)

- In this test, an applied voltage (60V for 6 hours) is used to drive a 3 percent sodium chloride solution into the depths of a core sample. The characteristics of the time-current curve give an indication of the permeability of concrete.
- Advantage: Test is capable of determining the susceptibility of concrete to chloride ion intrusion. Total time for testing is approximately one day.
- Disadvantage: Not as accurate as 90-day test.

Charge Passed (Coulombs)	Chloride Ion Penetrability
> 4,000	High
2,000 - 4,000	Moderate
1,000 - 2,000	Low
100 - 1,000	Very Low
< 100	Negligible

Figure B.16 Interpretation of Permeability Test Results

Carbonation

- Concrete provides a protective alkaline environment that protects embedded reinforcing from corrosion. This protective environment has a pH of about 12.5.
- Atmospheric gases, CO_2, can penetrate concrete and reduce the concrete alkalinity.
- The rate at which concrete carbonates is dependent on concrete quality, depth of rebar, ambient relative humidity, and degree of saturation. Fully saturated and dry concrete

are protected from carbonation, since they provide a barrier to gases or eliminate free water which is needed for the reaction, respectively.

- The depth of carbonation is approximately proportional to the square root of the time exposure.
- Carbonation reduces pH of concrete to 8.5, when fully carbonated.
- Phenolphthalein solution is applied to concrete to detect carbonation. When the surface turns pink, the pH is greater than 10. When the surface remains neutral, the pH has been reduced to less than 10 thereby increasing the likelihood that corrosion will initiate.

Advantages

- Easy and economical test

Disadvantages

- Requires samples of concrete, through core or other means to complete test

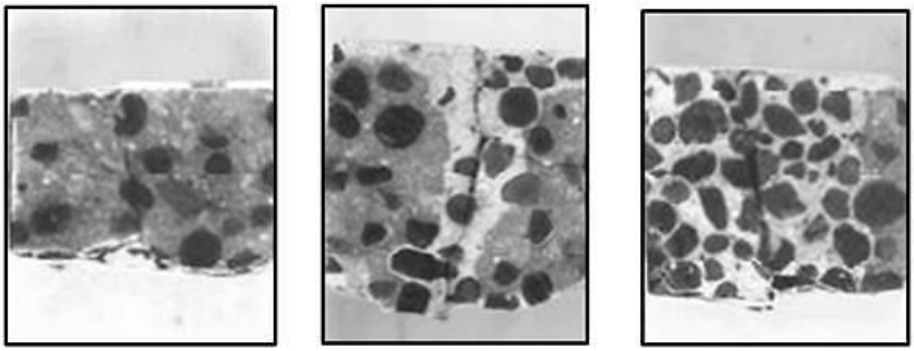


Figure B.17 Pink (darker) areas have pH > 10. Neutral areas are carbonated. Note carbonation along cracks, which provide some indication of crack age.

B.4.18 Ultrasonic Testing (UT): Evaluation of Steel Structures

Principles of UT

- Ultrasonic testing (UT) can be used on ferrous and nonferrous materials and often is suited for testing thicker sections accessible from one side only. In general, it can detect finer linear or planar defects than can radiography.
- UT makes use of mechanical vibrations similar to sound waves but of higher frequency. A beam of ultrasonic energy is directed into the object to be tested. This beam travels through the object with insignificant energy loss, except when it is intercepted and reflected by a discontinuity.
- The ultrasonic contact pulse reflection technique is used in UT. This system uses a transducer, which converts electrical energy into voltage that causes a crystal to vibrate mechanically. The crystal probe becomes the source of ultrasonic mechanical vibration. These vibrations are transmitted into the test piece through a coupling fluid, usually a film of oil, called a couplant.
- When the ultrasonic waves pulse strikes a discontinuity in the test piece, it is reflected back to its point of origin. Thus, the energy returns to the transducer. The transducer now serves as a receiver for the reflected energy.
- The initial signal, or main bang; the returned echoes from the discontinuities; and the echo of the rear surface of the test piece all are displayed by a trace on the screen of a cathode-ray oscilloscope. The detection, location, and evaluation of discontinuities become possible because the velocity of sound through a material is nearly constant, making distance measurement possible, and the relative amplitude of a reflected pulse is more or less proportional to the size of the reflector.
- One of the most useful characteristics of UT is its ability to determine the exact position of a discontinuity in a weld. This testing method requires a high level of operator training and competence and depends on establishing and applying suitable testing procedures.

Typical Applications

- Bridge pin inspection
- Penetration welds (plate girder flanges, circumferential welds in pipe, etc.)
- Pipe inspection for corrosion or damage
- Length and thickness measurements
- Plug and slot weld inspection

Advantages

- Nondestructive
- Volumetric
- Absence of hazard
- Versatile
- Single side access is often adequate
- Highly accurate

Disadvantages

- Skill/experience dependent
- Dynamic signal
- Surface condition critical
- Permanent record has limited value

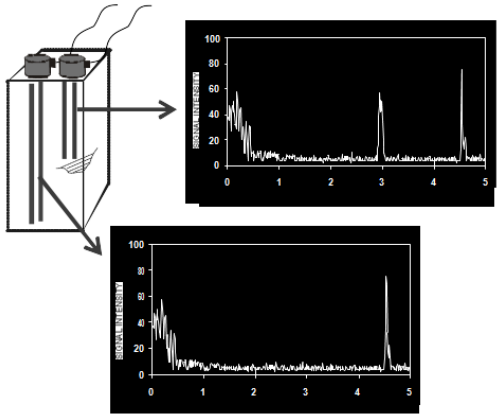


Figure B.18 Schematic of UT and Example Output Data

Figure B.19 UT Inspection of Bridge Hanger Pins (left), Bogie Shaft (middle), Sample CJP Weld (right)

B.4.19 Dye Penetrant Testing

Liquid Penetrant Testing (PT) is a widely used NDE technique. The test material is coated with a visible or fluorescent dye solution. Excess dye is removed from the surface, and then a developer is applied. The developer acts like a blotter and draws penetrant out of imperfections that are open to the surface. With visible dyes, the vivid color contrast between the penetrant and the developer makes the "bleed-out" easy to see. With fluorescent dyes, an ultraviolet lamp is used to make the bleed-out fluorescent, thus allowing the imperfection to be seen readily. Visible and fluorescent liquid penetrant examination reveals surface discontinuities on materials having nonporous qualities. PT consists of portable, economical equipment that can be easily used for inspections. A basic kit consists of a small grinder to remove rust inhibitors, a part cleaner, the dye penetrant, and a developer to facilitate the bleed out of excess dye from a discontinuity. It is fast, simple, and can cover large areas of inspection. Again, this test can only be performed on flaws that are free from debris and open to the surface. This method works well with both ferrous and nonferrous materials.

B.4.20 Magnetic Particle Testing of Welds: Evaluation of Steel Structures

Principles of MT

- Magnetic particle testing (MT) can be used on ferromagnetic materials to detect surface and near-surface linear discontinuities (i.e., cracks). The maximum depth of inspection is approximately 0.1 in.
- MT makes use of permanent or electro magnets for local magnetization of the surface being inspected. The magnets create a magnetic field. The direction of the magnetic field changes when it encounters a discontinuity. The change in magnetic field direction is often called magnetic flux leakage. After magnetizing the object, fine magnetic particles are then applied to the surface being inspected. The particles are attracted to the poles as well as locations of flux leakage resulting in visible indications of the surface of the object being inspected.
- MT equipment for civil engineering structures is generally handheld, runs off of 120 Volts, and can be AC or DC. DC generally provides a stronger magnetic field resulting in the ability to detect near-surface discontinuities. AC generally provides a weaker magnetic field but can produce greater excitation of the fine particles as they are applied to the test surface.
- Magnetic particles vary from 1/8 inch to 60 microns. The particles have high permeability (easily magnetized) and low retentivity (ability to stay magnetized). In other industries, the fine particles are often suspended in oil or paraffin to ensure movement of the particles to the location of flux leakage. However, most civil engineering applications use dry, colored particles.
- Cleaning and/or removal of paint and other debris may be required prior to testing. Cleaning should be performed using mechanical means rather than chemicals, since chemicals may mask some discontinuities.

Typical Applications

- Any welded or bolted connection where cracking is a concern
- Gusset plate inspections in bridges
- Base plate inspection of towers, light poles, signs, and other flexible or dynamically sensitive structures
- Transverse floor beam inspection in bridges
- Longitudinal stiffener plate inspection in bridges

Advantages

- Nondestructive
- Linear discontinuities
- Absence of hazard
- Versatile
- Single side access is often adequate
- Highly accurate
- Quick (excluding surface preparation)
- Requires moderate operator skill

Disadvantages

- Cannot detect subsurface discontinuities
- Magnetic field must intercept the larger dimension of the discontinuity
- Complex geometries can be problematic to inspect
- Skill/ experience dependent
- Surface condition critical
- No permanent record

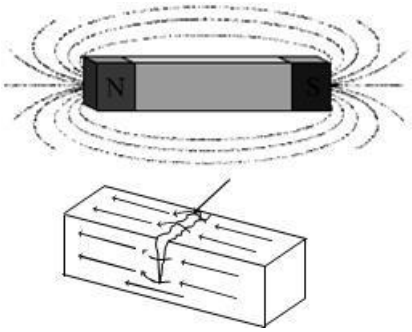


Figure B.20 Examples of Magnetic Fields

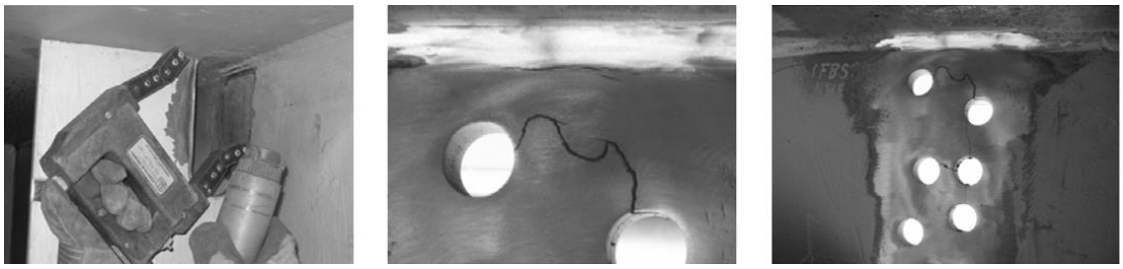


Figure B.21 Magnetic Particle Testing Showing: Magnetic particle yoke and particles in use after primer and paint removal (left); Crack after magnetic particle application (middle); Numerous cracks at a transverse floor beam (right)

B.5 AWPA Tables

The following tables were developed by the American Wood Preservers' Association (AWPA). They are used to interpret classification of timber types and timber uses.

Use Category	Service Conditions	Use Environment	Common Agents of Deterioration	Typical Applications
UC1	Interior construction Above ground Dry	Continuously protected from weather or other sources of moisture	Insects only	Interior construction and furnishings
UC2	Interior construction Above ground Damp	Protected from weather, but may be subject to sources of moisture	Decay fungi and insects	Interior construction
UC3A	Exterior construction Above ground Coated & rapid water runoff	Exposed to all weather cycles, not exposed to prolonged wetting	Decay fungi and insects	Coated millwork, siding and trim
UC3B	Exterior construction Above ground Coated & or poor water runoff	Exposed to all weather cycles, including prolonged wetting	Decay fungi and insects	Decking, deck joists, railings, fence pickets, uncoated millwork
UC4A	Ground contact or fresh water Non-critical components	Exposed to all weather cycles, normal exposure conditions	Decay fungi and insects	Fence, deck, and guardrail posts, crossties & utility poles (low decay areas)
UC4B	Ground contact or fresh water Critical components or difficult replacement	Exposed to all weather cycles, high decay potential including salt water splash	Decay fungi and insects with increased potential for biodeterioration	Permanent wood foundations, building poles, horticultural posts, crossties & utility poles (high decay areas)
UC4C	Ground contact or fresh water Critical structural components	Exposed to all weather cycles, severe environments, extreme decay potential	Decay fungi and insects with extreme potential for bio-deterioration	Land & freshwater piling, foundation piling, crossties & utility poles (severe decay areas)
UC5A	Salt or brackish water and adjacent mud zone Northern waters	Continuous marine exposure (salt water)	Salt water organisms	Piling, bulkheads, bracing
UC5B	Salt or brackish water and adjacent mud zone NJ to GA, south of San Fran	Continuous marine exposure (salt water)	Salt water organisms including creosote tolerant *Limnoria tripunctata*	Piling, bulkheads, bracing
UC5C	Salt or brackish water and adjacent mud zone South of GA & Gulf Coast	Continuous marine exposure (salt water)	Salt water organisms including creosote tolerant *Martesia, Sphaeroma*	Piling, bulkheads, bracing
UCFA	Fire protection as required by codes Above ground Interior construction	Continuously protected from weather or other sources of moisture	Fire	Roof sheathing, roof trusses, studs, joists, paneling
UCFB	Fire protection as required by codes Above ground Exterior construction	Subject to wetting	Fire	Vertical exterior walls, inclined roof surfaces or other construction which allows water to quickly drain

Figure B.22 AWPA Classification Table for Lumber

USE	AWPA STANDARD	OIL-TYPE PRESERVATIVES Minimum Retentions – Pounds Per Cubic Foot			WATERBORNE PRESERVATIVES Minimum Retentions – Pounds Per Cubic Foot						
	Use Category System	Copper Naphthenate[1]	Creosote[2]	Pentachlorophenol[3]	ACQ[4]	ACZA[5]	CA-C[6]	CCA[7]	SBX[8]	EL2[15]	PTI[16]
FENCING											
Pickets, slats, trim	3A, 3B	0.055	8.0	0.50	0.15	0.25	0.06	NP	NL	0.019	0.018[17]
Posts, sawn	4A	NL	10.0	0.50	0.40	0.40	0.15	NP	NL	NL	NL
Posts, round	4A	0.055	8.0	0.40	0.40	0.40	0.15[12]	NP	NL	NL	NL
Rail	3A, 3B	0.055	8.0	0.50	0.15	0.25	0.06	NP	NL	0.019	0.018[17]
HIGHWAY MATERIAL											
Lumber and timbers for bridges, structural members, decking, cribbing and culverts	4B	0.075	10.0	0.50	0.60	0.60	0.31[11]	0.60	NL	NL	NL
Structural lumber and timbers:											
• In saltwater use and subject to marine borer attack	5A, 5B, 5C	NL	25.0	NL	NL	2.50	NL	2.50	NL	NL	NL
• Piles, foundation, land and fresh water use	4C	0.10 – 0.14	12.0 – 17.0	0.60 – 0.85	NL	0.80 – 1.0	NL	0.80 – 1.0	NL	NL	NL
• Piling in saltwater use and subject to marine borer attack	5A, 5B, 5C	NL	16.0 – 20.0	NL	NL	1.50 – 2.50	NL	1.50 – 2.50	NL	NL	NL
• Posts: Round, half-round, quarter-round (General const. – fence posts, sign posts, handrails)	4A	0.055	6.0 – 8.0	0.40	0.40	0.40	0.15	0.40	NL	NL	NL
• Posts: Round, half-round, quarter-round (Guardrails,spacer blocks, critical structural members)	4B	0.069	10.0	0.50	0.50	0.50	0.31	0.50	NL	NL	NL
• Posts: Sawn (General const. – fence posts, sign posts, handrails)	4A	0.06	10.0	0.40	0.40	0.40	0.15	0.40	NL	NL	NL
• Posts: Sawn (Guardrails,spacer blocks, critical structural members)	4B	0.075	10.0	0.50	0.50	0.50	0.31	0.50	NL	NL	NL
LUMBER											
Above ground	3B	0.04	8.0	0.40	0.15	0.25	0.06	NP	NL	0.019	0.018[17]
Ground contact and freshwater use	4A	0.06	10.0	0.50	0.40	0.40	0.15	NP	NL	NL	NL
MARINE LUMBER AND TIMBERS											
Members above ground and out of water but subject to saltwater splash	4B, 4C	0.06 – 0.75	10.0 – 12.0	0.50 – 0.60	0.60	0.60	0.31	0.60	NL	NL	NL
In brackish or saltwater use and subject to marine borer attack	5A, 5B, 5C	NL	25.0	NL	NL	2.50	NL	2.50	NL	NL	NL
PILES											
Foundation, land and freshwater use (round)	4C	0.10 – 0.14	12.0 – 17.0	0.65 – 0.85	0.80	0.80 – 1.0	NL	0.80 – 1.0	NL	NL	NL
Marine (round) in salt or brackish and subject to marine borer attack	5A, 5B, 5C	NL	16.0 – 20.0	NL	NL	1.50 – 2.50	NL	1.50 – 2.50	NL	NL	NL
Marine, dual treatment (round) for maximum protection	5B, 5C	NL	20.0	NL	NL	1.0	NL	1.0	NL	NL	NL
Sawn timber piles	4B, 4C	0.075	10.0 – 12.0	0.50	0.60	0.60 – 0.80	NL	0.60 – 0.80	NL	NL	NL
PLYWOOD											
Sub-floor, damp, above ground	2	0.04	8.0	0.40	0.15	0.25	0.06	0.25	0.25	0.019	0.013
Exterior, above ground	3B	NL	8.0	0.40	0.15	0.25	0.06	0.25	NL	0.019	0.018[17]
Ground contact	4A	NL	10.0	0.50	0.40	0.40	0.15	0.40	NL	NL	NL
Out of contact with ground and continously protected from liquid water	2	NL	NL	NL	0.15	NL	NL	NL	0.25	0.019	0.013
Marine	5A, 5B, 5C	NL	25.0	NL	NL	2.50	NL	2.50	NL	NL	NL

Figure B.23 AWPA Standard Use Classification System

List of Acronyms

ASR	Alkali Silica Reactivity
AANC	Aging Aircraft Nondestructive Inspection Validation Center
ACQ	Alkalkine Copper Quaternary
AC	Alternating Current
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AGMA	American Gear Manufacturer's Association
ASTM	American Society for Testing and Materials
AWPA	American Wood Protection Association
ACZA	Ammonical Copper Zinc Arsenate
ACB	Articulated Concrete Block
AC	Asphalt Concrete
APJ	Asphaltic Plug Joints
ADT	Average Daily Traffic
BIRM	Bridge Inspectors Reference Manual
BMS	Bridge Management System
CSA	Calcium Sulfonate Alkyd
CE	Carbon Equivalent
CIP Concrete	Cast-In-Place Concrete
CP	Cathodic protection
CCA	Chromated Copper Arsenate
CERCLA	Comprehensive Environmental Response Compensation and Liability Act
CPC	Corrosion Preventing Compounds
CMP	Corrugated Metal Pipe
CPM	Critical Path Method
CFM	Cubic Feet per Minute
CIPP	Cured In Place Pipe
DEF	Delayed Ettringnite Formation
DOT	Department of Transportation
DC	Direct Current
DTI	Direct Tension Indicator
DFT	Dry Film Thickness
EPA	Environmental Protection Agency
EPDM	Ethylene Propylene Diene Monomer
EP	Extreme Pressure
FFT	Fast Fourier Transform
FHWA	Federal Highway Administration
FIFRA	Federal Insecticide, Fungicide and Rodenticide Act

FRP	Fiber Reinforced Polymer
GPS	Global Positioning System
GGBFS	Ground Granulated Blast-Furnace Slag
GPR	Ground penetrating radar
HEPA	High Efficiency Particulate Air
HMWM	High Molecular Weight Methacrylate
HP WC	High Pressure Water Cleaning
HP WJ	High Pressure Water Jetting
HS	High Strength
HRWR	High-Range Water Reducers
HVLP	High-Volume, Low-Pressure
HEC	Hydraulic Engineering Circular
IE	Impact echo
IR	Impulse Response
IT	Infrared thermography
I/O	Input/Output
ITS	Intelligent Traffic Systems
ICRI	International Concrete Repair Institute
ISO	International Standards Organization
JOC	Job Order Contract
LVL	Laminated Veneer Lumber
LRFD	Load and Resistance Factor Design
MBE	Manual for Bridge Evaluation
MFP	Magnetic Field Perturbation Testing
MFL	Magnetic flux leakage
MT	Magnetic-Particle Testing
MMS	Maintenance Management Systems
MASH	Manual for Assessing Safety Hardware
MUTCD	Manual for Uniform Traffic Control Devices
MSDS	Material Safety Data Sheets
MBE	MBE standards
MSE	Mechanically Stabilized Earth
MIT	Methyl IsoThiocyanate
MIO	Micaceous Iron Oxide
MCU	Moisture Cured Urethane
NACE	National Association of Corrosion Engineers
NBIS	National Bridge Inspection Standards
NBIS	National Bridge Inventory
NCHRP	National Cooperative Highway Research Program
NEMA	National Eletric Motors Association
NHI	National Highway Institute
NHS	National Highway System
NLGI	National Lubricating Grease Institute

NDEVC	NDE Validation Center
NYSDOT	New York State Department of Transportation
NDE	Non-Destructive Evaluation
NDT	Non-Destructive Testing
OSHA	Occupational Safety and Health Administration
OBPA	Ogdensburg Bridge and Port Authority
AL	OSHA Action Level
PEL	OSHA Permissible Exposure Limit
PC	Personal Computer
PTFE	PolyTetraFlourEthylene
PVC	PolyVinyl Chloride
PCC	Portland Cement Concrete
PSI	Pounds Per Square Inch
PM	Preventive Maintenance
PDS	Product Data Sheet
PLC	Programmable Logic Controller
QPL	Qualified Product List
QA	Quality Assurance
QC	Quality Control
QMP	Quality Management Plan
RBME	Regional Bridge Maintenance Engineer
RCRA	Resource Conservation and Recovery Act
SDS	Safety Data Sheets
SSD	Saturated Surface Dry
ASM	Society for Materials Scientists and Engineers
SSPC	Society for Protective Coatings
SAE	Society of Automotive Engineers
SASW	Spectral Analysis of Surface Waves
SOP	Standard Operating Procedure
SHRP	Strategic Highway Research Program
SHM	Structural Health Monitoring
SBA	Submerged Bulk Anode
TSZ	Thermally Sprayed Zinc
TSP	Trisodium Phosphate
UHP WJ	Ultra High Pressure Water Jetting
UV	Ultra Violet
UPE	Ultrasonic Pulse Echo
UPV	Ultrasonic Pulse Velocity
USW	Ultrasonic Surface Waves
UT	Ultrasonic Testing
USACE	United States Army Corps of Engineers
VARTM	Vacuum-Assisted Resin-Transfer Molding
VDOT	Virginia Department of Transportation

VI	Viscosity Index
VI	Visual Inspection
VOC	Volatile Organic Compounds
WFT	Wet Film Thickness

English to Metric Conversion Table

Multiply	By	To Get	Multiply	By	To Get
Acres	0.404 686	Hectares	Ounce (force)	0.278 013 9	Newtons = N
Board Feet	0.002 359 74	Cubic meter	Pint (liquid)	0.473 176	Liters = l
Cubic Feet	0.028 316 85	Cubic meter	Pint (dry)	0.550 61	Liters = l
Cubic Yard	0.764 554 9	Cubic meter	Pound (weight)	0.453 592 37	Kilogram
Feet	0.304 8	Meters	Pound (force)	4.448 222	Newtons = N
Footcandles	10.763 91	Lux=lumens/m^2	Pound/square feet	47.880 26	Pascal = N/m^2
Ft.-lb_f	1.355 818	Nm=Joule	Pound/sq.in	6.894 757	Kilopascals
Gallon (US)	3.785 412	Liter	Quart (liquid)	0.946 352 9	Liters
Horsepower*	745.699 9	Watt=J/sec	Square feet	0.092 903	Square meter
*Horsepower =	550 ft-lb_f/sec		Square inch	645.16	Square mm
Inch	25.4	Millimeters	Square mile	258.998 8	Hectares
Inch-$pound_f$	0.112 984 8	Nm=Joule	Square mile	2.589 988	Square km
Kips	4.448 222	Kilonewton	Square yard	0.836 127 4	Square meter
Kips/in^2	6.894 757	Megapascal	Ton (short)	0.907 184 7	Metric ton
Miles (US)	1.609 344	Kilometer	Ton (short)	907.184 7	Kilogram = kg
Ounce (weight)	28.349 52	Grams	Ton (short)	8896.444	Newtons = N
Ounce (liquid)	29.573	Milliliter	Yards	0.914 4	Meters = m

For Temperature Conversion Use °C = 5/9(°F - 32)

Metric to English Conversion Table

Multiply	By	To Get	Multiply	By	To Get
Cubic Meter	1.308 0	Cubic yard	Milliliter	0.033 814	Fluid ounce
Cubic Meter	35.314 7	Cubic foot	Liter/m	0.080 52	Gal/ft
Cubic Meter	61,024	Cubic inch	Liter/m^2	0.220 88	Gal/sq. yd
Cubic Meter	264.172	Gallon	Lux	0.092 902	Ft-candle
Gram	0.035 274	Ounce (weight)	Meter	1.093 6	Yard
Gram	0.002 204 6	Pound (weight)	Meter	3.280 84	Foot
Kilogram	35.274	Ounce (weight)	Millimeter	0.039 370	Inch
Kilogram	2.204 623	Pound (weight)	Kilometer	0.621 37	Mile
Kilogram	0.002 204 6	Kip	Micrometer	0.039 370 1	Mil
Kilogram	0.001 102 3	Ton	Newton	0.224 81	Pound(f)
Metric ton	1.102 3	Ton	Kilonewton	0.224 81	Kip(f)
Hectare	2.471 04	Acre	Pascal	0.020 885	Pound/sq. ft.
Hectare	107,639	Square feet	Kilopascal	0.145 04	Pound/sq. inch
Hectare	11,959.9	Square yard	Megapascal	0.145 04	Kips/sq. inch
Hectare	0.003861 02	Square mile	Square meter	1.195 99	Square yard
Joule	0.737 56	Foot pound	Square meter	10.763 9	Square feet
Joule	8.849 5	Inch pound	Square millimeter	0.001 55	Square inch
kg/m^3	1.685 55	Pounds/cubic yard	Square kilometer	0.386 102	Square mile
kg/m^3	0.062 428	Pounds/cubic feet	Watt (J/sec)	0.001 341	Horsepower
km/hr	0.621 37	Miles per hour	Watt (J/sec)	0.737 56	Ft-lb/second
Liter	0.264 17	Gallon			
Liter	1.056 7	Quart			
Liter	2.113 4	Pint			
Liter	33.814	Fluid ounce			

For Temperature Conversion Use °F =9/5 °C + 32